CONCRETE
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CONSTRUCTIONAL ENGINEERING

INCLUDING PRESTRESSED CONCRETE

JUNE 1956 VOL. LI. NO. 6

FIFTY-FIRST YEAR OF PUBLICATION

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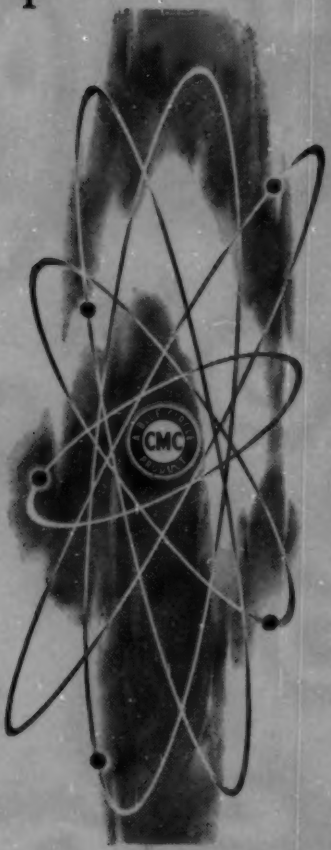
Concrete plays a major part in the construction of Britain's atomic factories



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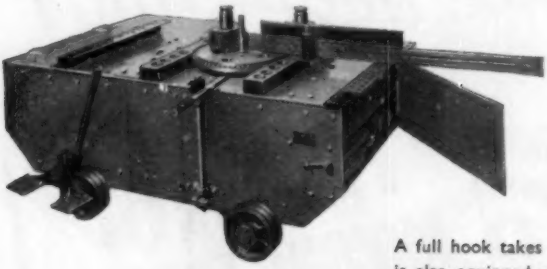

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This is a single-disc machine which is specially designed for an extraordinarily high rate of bending bars up to 1¼ in. diameter. A full hook takes three seconds' bending time. It is also equipped with accessories for bending angle loops in one operation on mild steel reinforcement up to 1 in. diameter.

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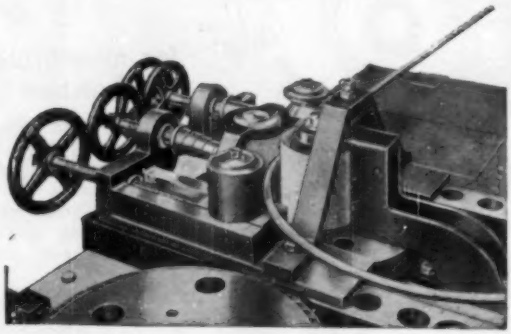

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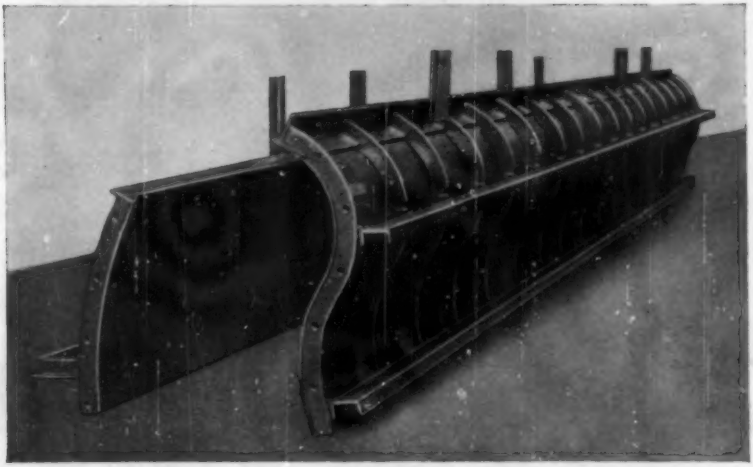

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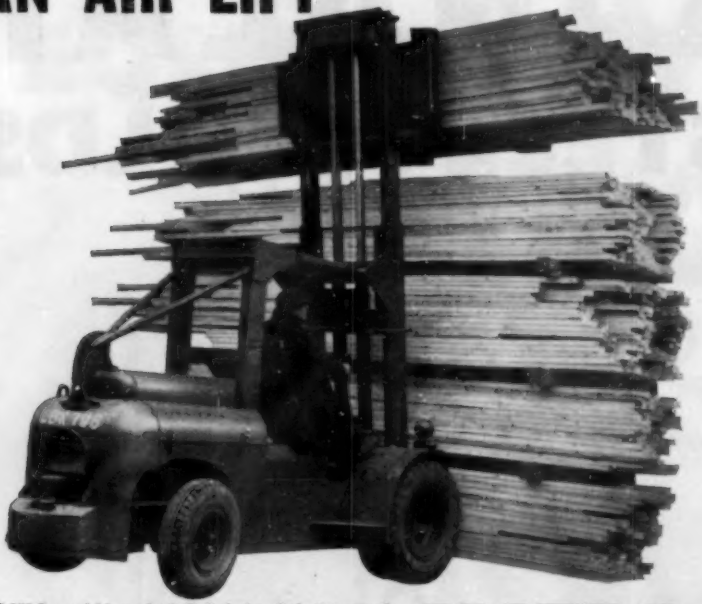

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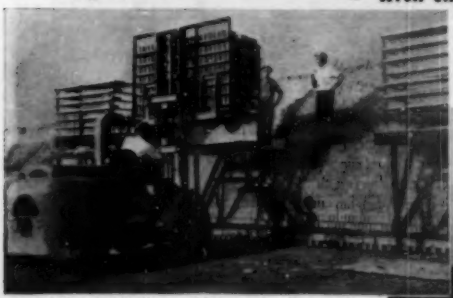

Matbro 6,000 lbs model in use by Eastwoods Ltd., Sittingbourne

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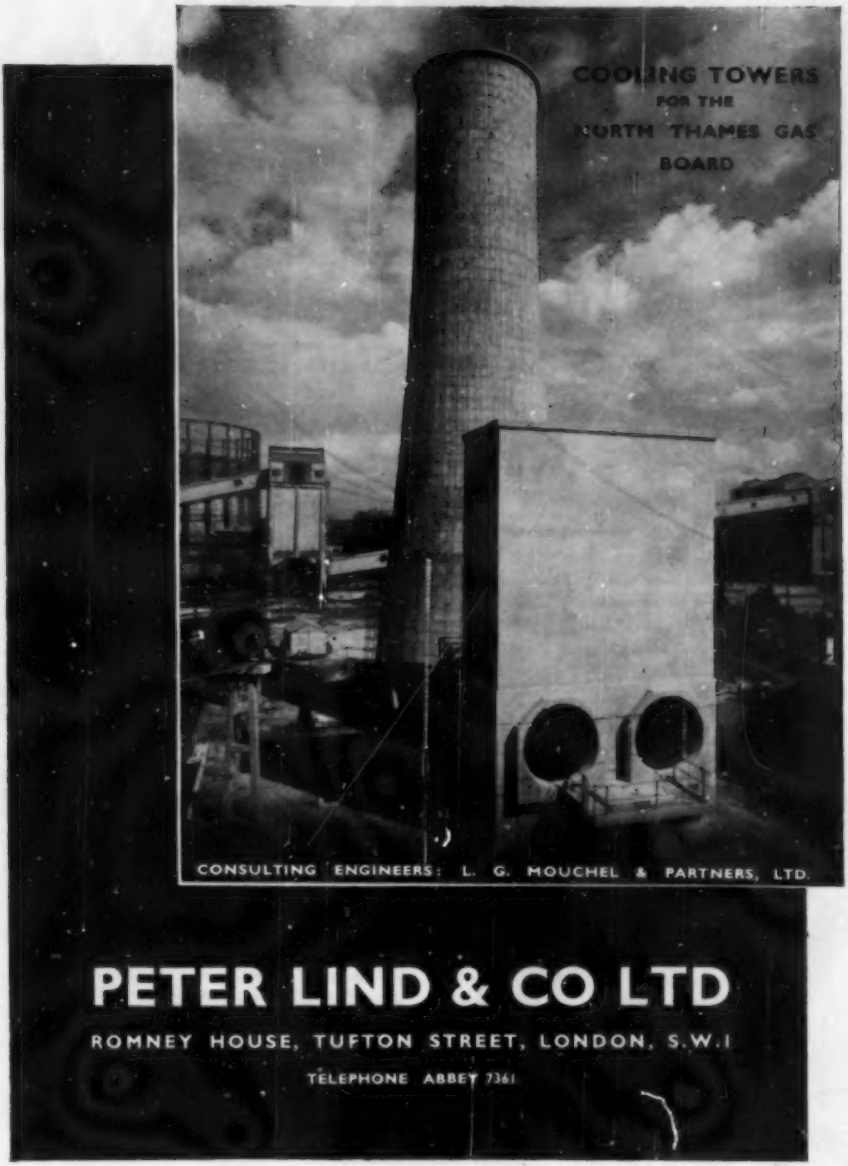

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AGLITE

Statement by the Chairman of the Butterley Company.

You may recall that—even before 1946—coal, although the main interest of our Company, was by no means the only one. Since nationalisation, we have perforce concentrated on the expansion of those assets remaining in our charge—largely by adopting new processes and techniques. For example, two years ago we acquired a licence to manufacture the best American plant for making pure Oxygen on site—so successfully that we now have orders for over one-and-a-half million pounds.

Similarly, the growth of our specialised brick manufactures encourages us in our promotion of a lightweight building aggregate, Aglite. This has been well proved in the United States; its exceptional properties, notably a strength-to-weight ratio not hitherto achieved in Britain, place it far ahead of anything as yet made available to this country's architects and builders.

We have not embarked on this adventure without thorough investigation. 70 tons of our indigenous material went to the United States for processing, and proved on test to produce an Aglite identical with the American material. Our own pilot plant, now running for over a year, has been stringently tested: samples from it are now with the National Building Research Station; and the first production machine, in Derbyshire, is scheduled to begin operating during the Autumn. The siting of later machines will ensure a national distribution of Aglite.

Aglite, I am sure, will make a notable contribution to the solution of many problems which face the Building Industry today.

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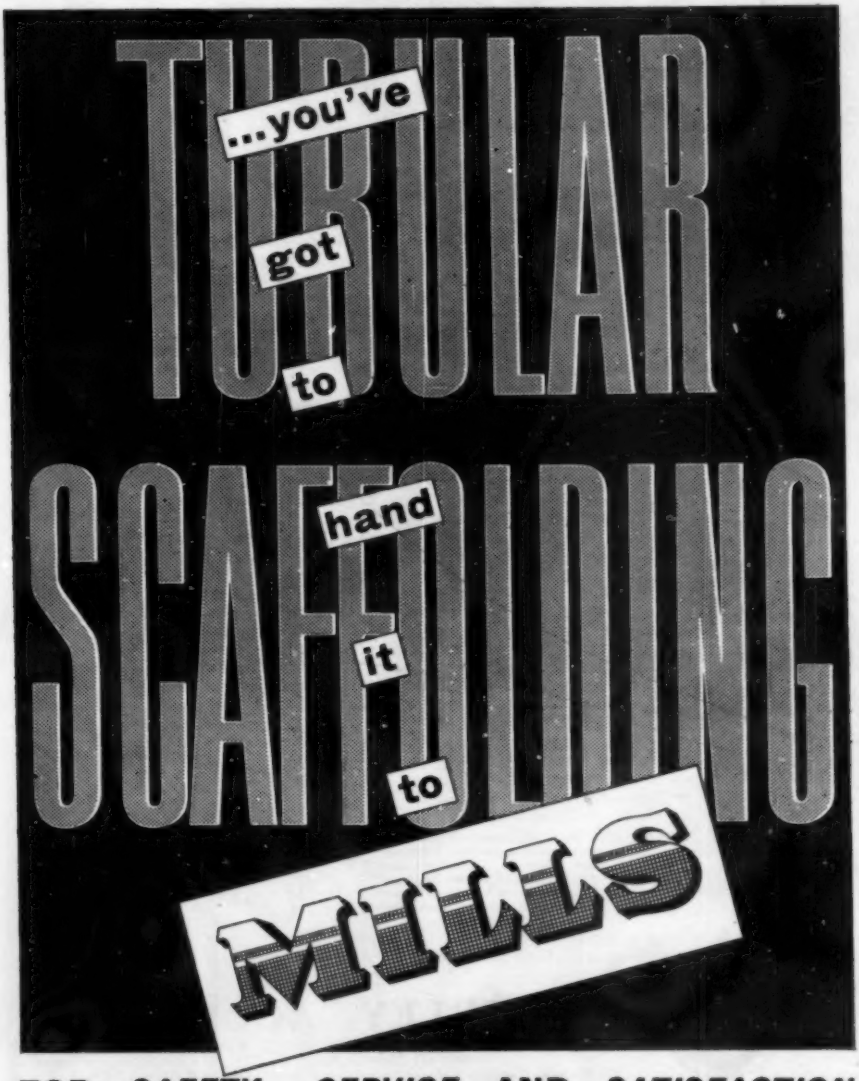

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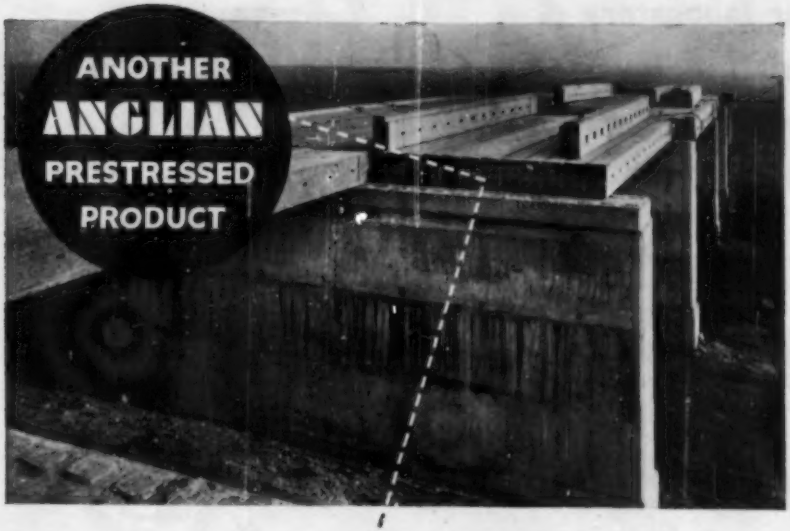

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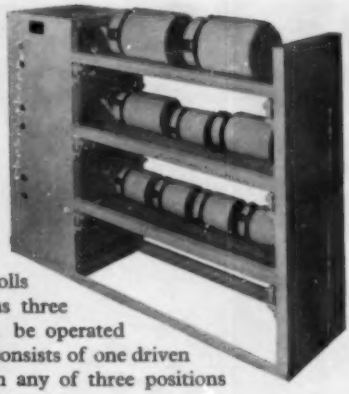

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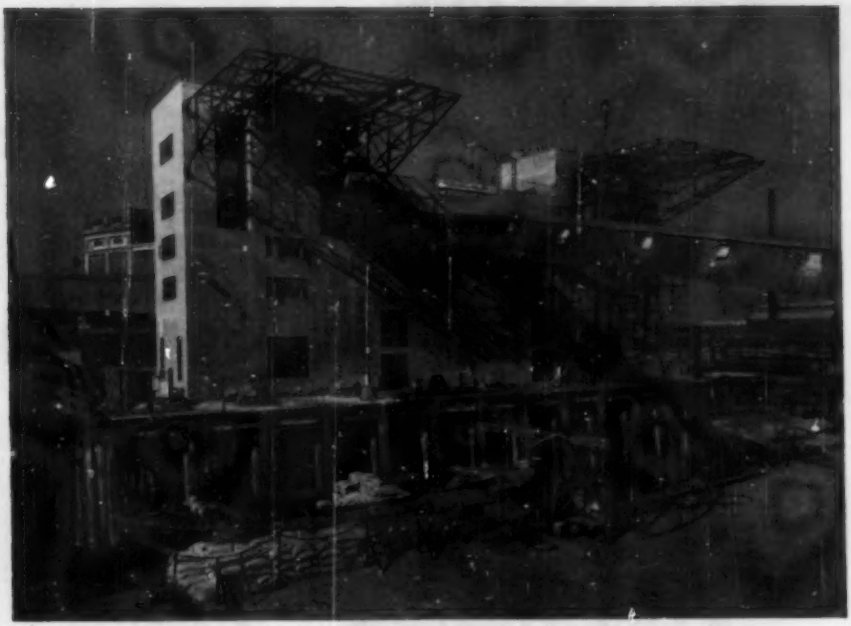

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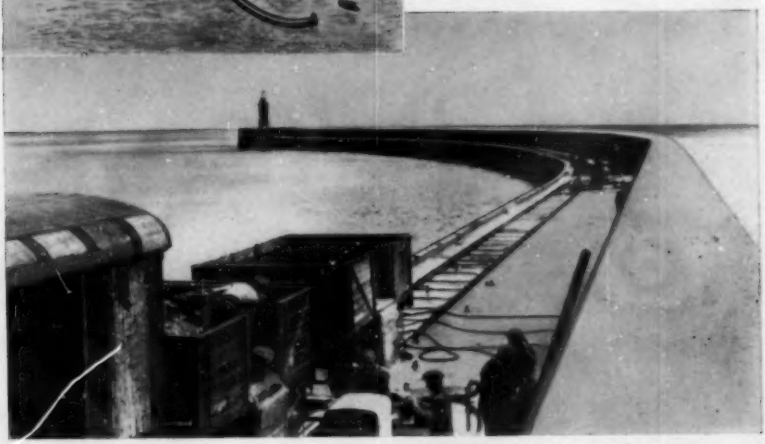

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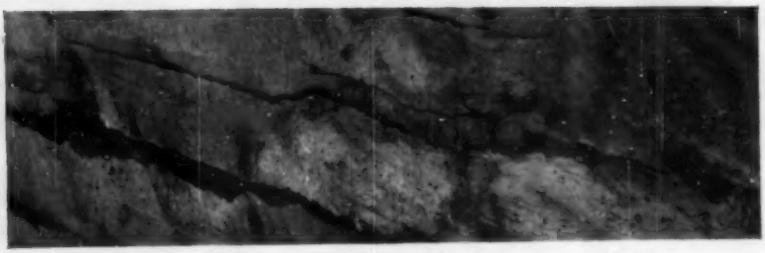
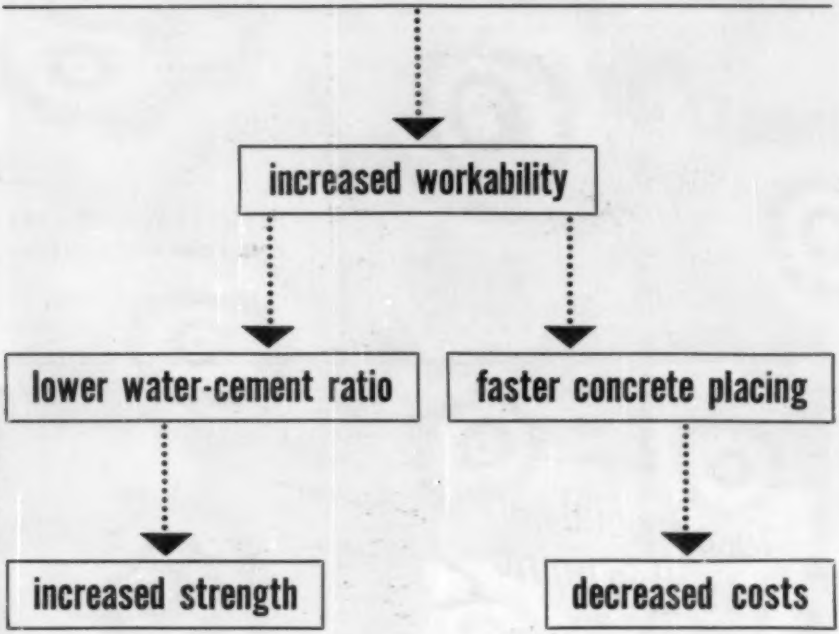


FEBCRETE

air entrained concrete



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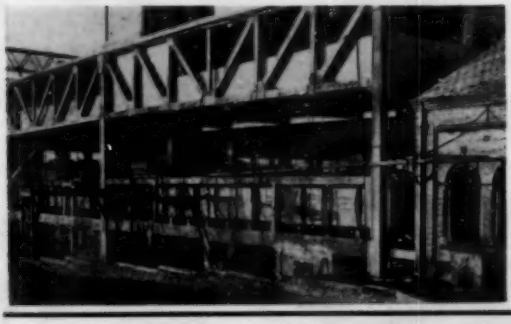
The Repaired Structure—BEFORE and AFTER



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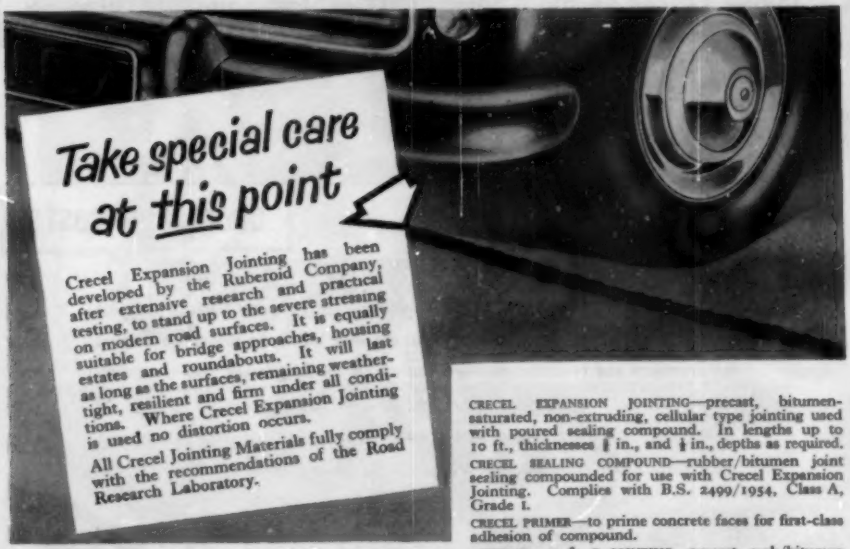

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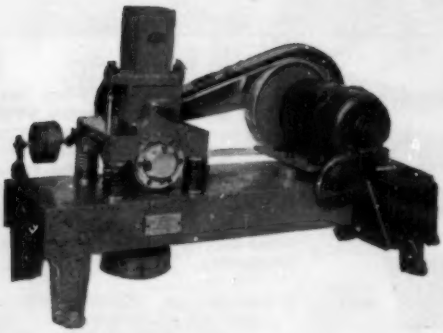

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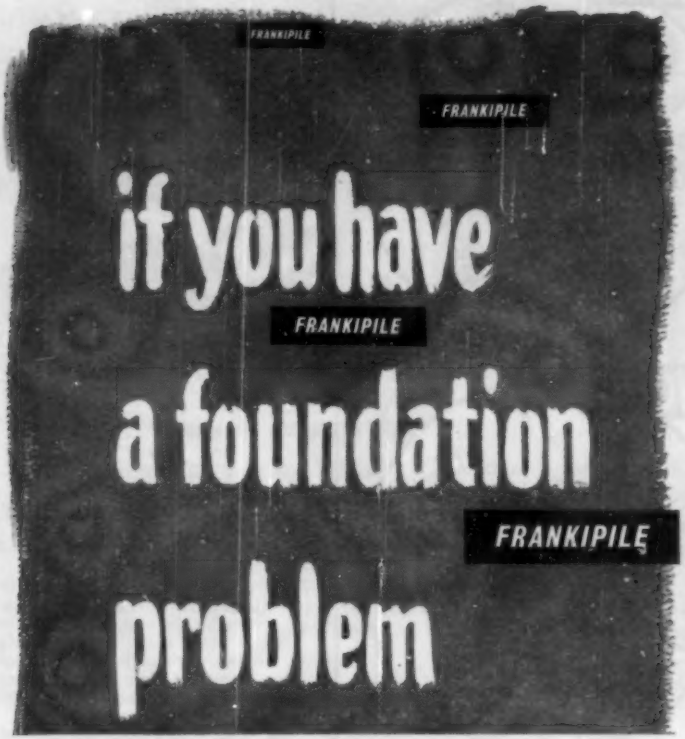

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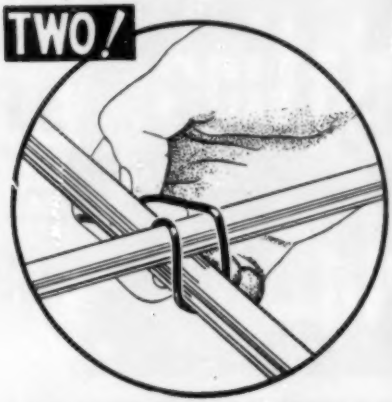

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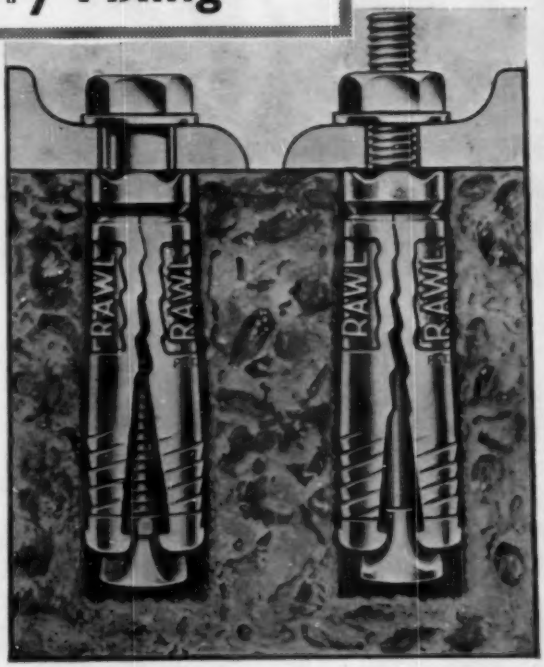

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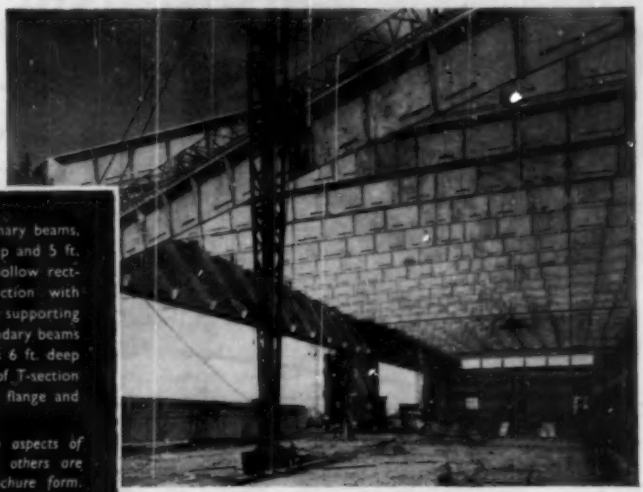

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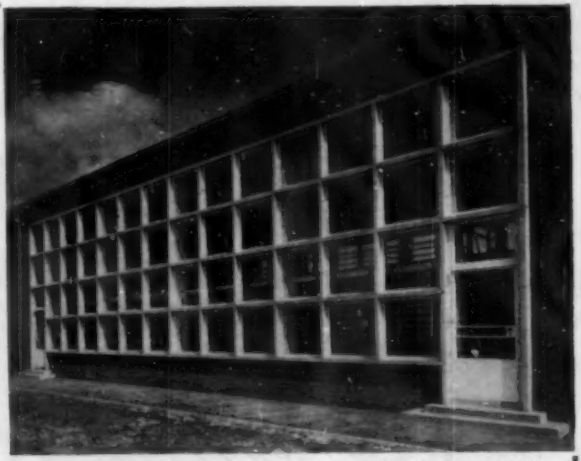

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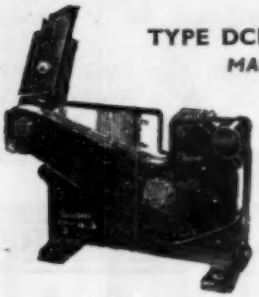

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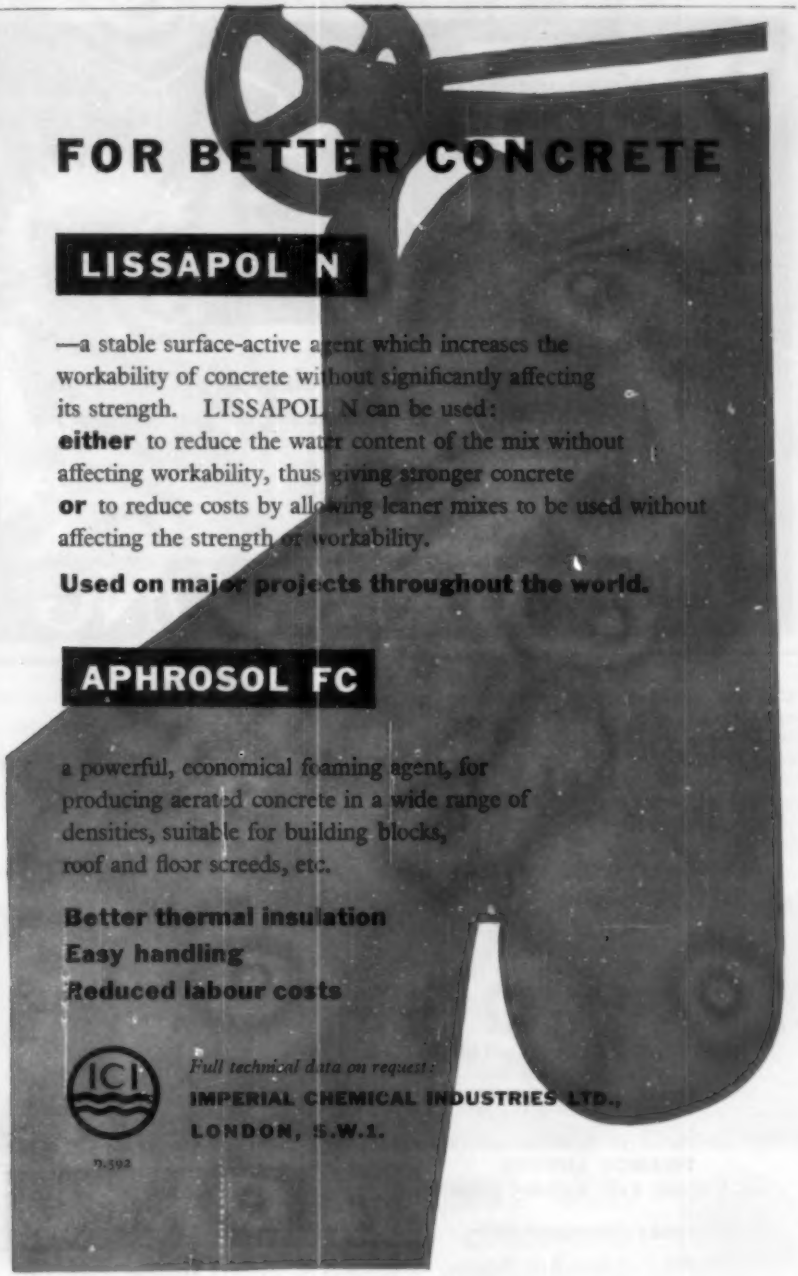

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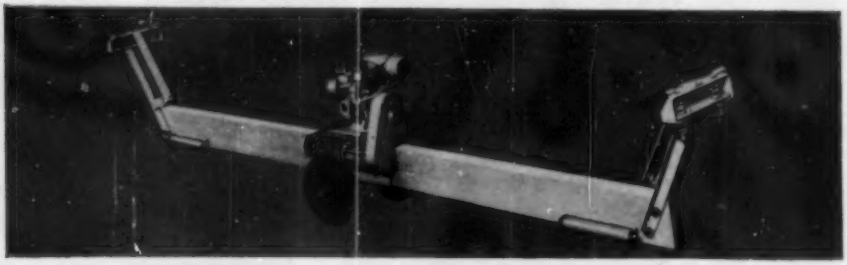

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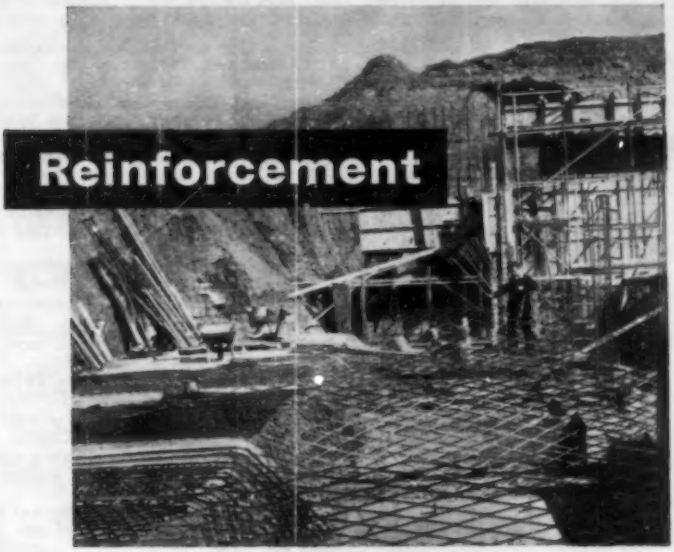

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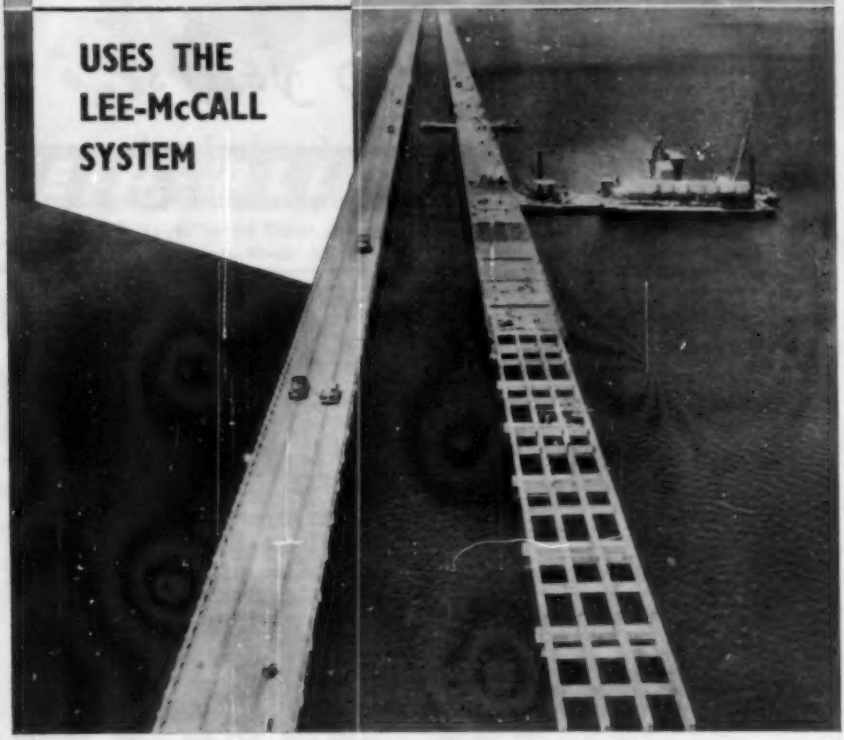

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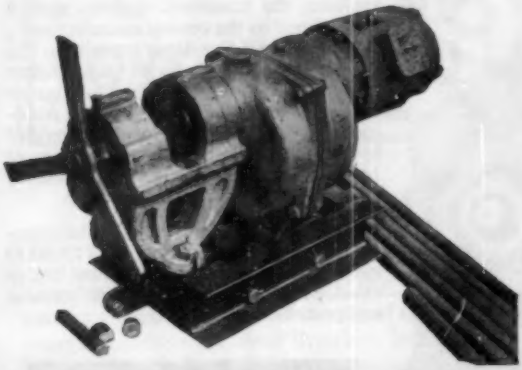

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CONCRETE AND CONSTRUCTIONAL ENGINEERING

INCLUDING PRESTRESSED CONCRETE

Volume LI, No. 6. LONDON, JUNE, 1956.

EDITORIAL NOTES

The late Oscar Faber.

THE death of Dr. Oscar Faber, C.B.E., which occurred suddenly as the result of a heart attack on May 7, at the age of 69, marks the end of an era in the development of concrete in Great Britain. In the early days of the century reinforced concrete was looked upon with prejudice by many architects, and during this period most reinforced concrete work was done by specialist firms who used various systems in some of which proprietary rights were claimed. At the end of the first world war some of the younger engineers started in practice as consulting engineers, and were the founders of a new body of professional engineers specialising in reinforced concrete. Of these young consultants Oscar Faber was the undoubted leader, and he lived to see reinforced concrete generally accepted as one of the most important materials of construction. In 1919, when he started in practice as a consulting engineer in the City of London, he devoted much time by lectures and other means to gain the confidence of architects in the material. He had considerable gifts as an artist, especially in water colours, and as a musician, and these were of much help to him in his relations with architects. In the early 1920's, for example, he composed the music and conducted the orchestra at the annual pantomime of the Architectural Association.

His practice rapidly grew, and he was responsible for very many important works, including the rebuilding of the Bank of England, South Africa House, India House, and the new Church House in London, to mention only a few. He was also responsible for many large industrial structures in Shanghai, Hongkong, and elsewhere in the East as well as in Great Britain. In 1935 he was awarded an Honorary D.C.L. of Durham University in recognition of his work in underpinning and restoring Durham Castle. During the second world war he did a great deal of work for the Government, and was consulted by Sir Winston Churchill in connection with the Mulberry Harbour project. One of his most notable works since the war was the rebuilding of the House of Commons, and for this he was promoted to C.B.E. He was an untiring worker, and in spite of his large practice he found time to act as chairman of the Code of Practice Committee for Reinforced Concrete.

Faber was the son of a Danish Government official in London. He had a commanding presence and this, together with his authoritative position in reinforced concrete, at times gave the impression that he would brook no opposition

or criticism of his ideas. He was, however, always eager to see the lighter side of life—as, for example, when a building he had designed met with so much praise from architectural critics that he applied for membership of the Royal Institute of British Architects.

Faber's connection with this journal started in 1920, when he was appointed consulting editor after the death of the first editor. (A portrait is on p. 394.)

Mr. P. G. Bowie, who was at college with Faber and whose name is linked with Faber's as a joint author of " Reinforced Concrete Design ", writes as follows :

No one who knew Oscar Faber could fail to be impressed by his strength, ability, and integrity—qualities developed by a lifetime of effort to fit himself for the pre-eminent position he attained in structural engineering. As a student at what is now the Imperial College of Science and Technology, his hereditary gifts of energy and culture led him to value highly achievement and independence, and in taking the most desirable, but at the time unusual, fourth year course he was able to get a clearer understanding of the opportunities available to a well-trained engineer.

He realised the possibilities of such a comparatively new material as reinforced concrete and, relying almost entirely on his own efforts, set his course to acquire whatever knowledge was necessary to specialise in its development. Then, as now, practical experience was an essential part of an engineer's training, and he was fortunate in obtaining this as resident engineer under the late C. Percy Taylor during the construction of jetties and other cement-handling works. This insight into the use and manufacture of cement was followed by a period with the Indented Bar & Concrete Engineering Co., Ltd., where he gained experience of the detailing and design of a great variety of structures introducing some of the first applications of the theory of bending in columns. The outcome of this work was his book " Reinforced Concrete Design ", published in 1910, for he fully appreciated that it was as desirable in engineering as in the administration of justice that it should be " seen to be done ".

Not content with such activities, Faber's abounding energy led him to improve his gift of clear expression by teaching at the Northern Polytechnic, where, realising the need for an engineer to reduce uncertainties by experiment and research, he began the studies on shearing stresses in reinforced concrete beams that brought him a well-deserved Doctorate of Science. At each stage in his career one can trace Faber's fixed intention to acquire whatever knowledge and experience would fit him to undertake and carry out the functions of a consulting engineer. His last step before taking up private practice was to join the staff of Messrs. Trollope and Colls, Ltd., where he gained an insight into commerce and the organisation of contracts that was so helpful to him in later life.

The first world war set a period to our work together, and my last recollections of that time include our joining the tail end of a long recruiting queue at the headquarters of the Honourable Artillery Company before he was called on by the Admiralty to consider the production of non-magnetic mines. For his services during the war he was awarded an O.B.E.

By his sudden death I have lost a friend of fifty years, with whom I have worked at various times and for various purposes and, though in later years our paths through life have differed, neither time nor distance affected our friendship or lessened my admiration of his great achievements.

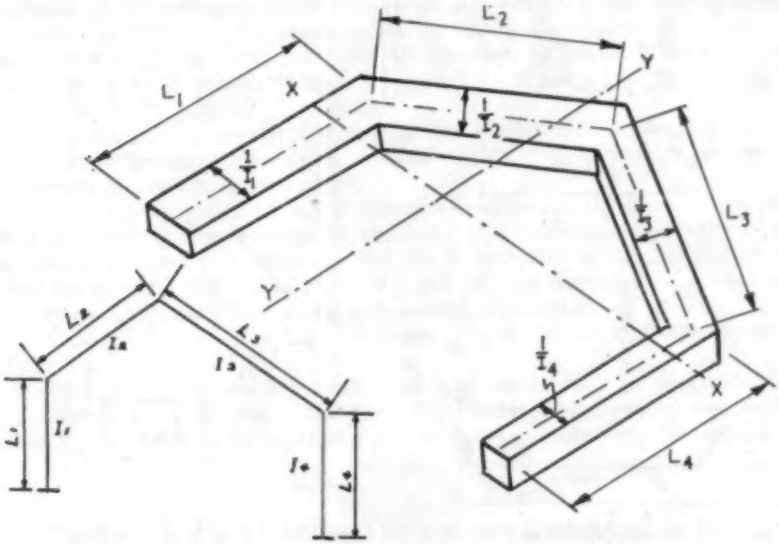
Analysis of Gable Frames by the Column Analogy Method.

By E. S. E. LYLE, B.Sc., A.M.I.C.E.

GABLE frames of one bay can be quickly analysed by means of the method of column analogy.

GENERAL CASE.—The frame shown in *Fig.* 1 consists of two columns rigidly fixed at their bases and two sloping rafters. The moment of inertia of the four members are I_1, I_2, I_3, and I_4, and their lengths are L_1, L_2, L_3, L_4. First, the frame is represented as the cross section of a short column (*Fig.* 2). The geometry of the centre-lines of the members of the analogous column is the same as that of the frame, and the width of each member is represented by $\frac{I}{I}$ where I is the moment of inertia of the corresponding member of the frame. The area of each member of the analogous column is then $a = L \times \frac{I}{I}$. The centroid of the cross section of the column is found, and through this point two convenient axes at right angles are drawn. The following properties of the cross section of the column are determined: A, the area; I_X, the moment of inertia about axis Y—Y; I_Y, the moment of inertia about axis X—X; I_{XY}, the product of inertia of the area about the two axes. The last term is necessary only when the frame is not symmetrical about the vertical axis either geometrically or with regard to its elastic properties.

Consider any system of loading. By severing the frame or introducing



Figs. 1 and 2.

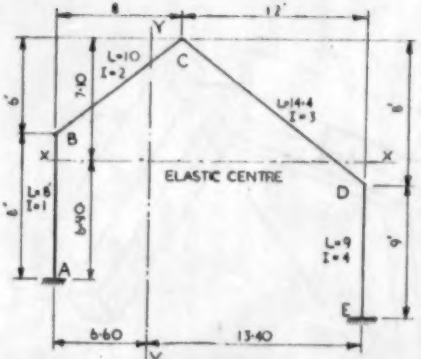
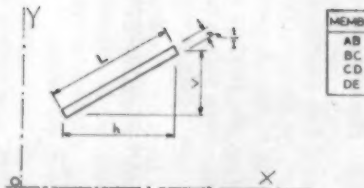


Fig. 3.



$\Sigma a = L\frac{1}{I}$ $i_x = \frac{1}{12}aH^2$ $i_y = \frac{1}{12}av^2$ $i_{xy} = \frac{1}{12}ahs$

N.B. i_{xy} + FOR SLOPE / − FOR SLOPE \

Fig. 4.

TABLE I.

MEMBER	L	$\frac{1}{I}$	a	x_1	y_1	ax_1	ay_1
AB	8·0	1·00	8·00	0	4·0	0	32·0
BC	10·0	0·50	5·00	4·0	11·0	20·0	55·0
CD	14·4	0·33	4·80	14·0	10·0	67·5	48·0
DE	9·0	0·25	2·25	20·0	1·5	45·0	3·4

$A = \Sigma a = 20{\cdot}05$ $\Sigma ax = 132{\cdot}5$ $\Sigma ay = 138{\cdot}4$

$\bar{x} = 6{\cdot}60$ $\bar{y} = 6{\cdot}90$

TABLE II.

MEMBER	a	x	y	ax^2+i_x	ay^2+i_y	$axy+i_{xy}$
AB	8·00	−6·60	−2·90	+349	+67	+153
				0	+43	0
BC	5·00	−2·60	+4·10	+34	+84	−53
				+27	+15	+20
CD	4·80	+7·40	+3·10	+263	+46	+110
				+58	+28	−38·5
DE	2·25	+13·40	−5·40	+405	+66	−163
				0	+15	−0

$I_x = 1113$ $I_y = 382$ $I_{xy} = 285$

$\frac{I_{xy}}{I_y} I_{xy} = 2$ $I'_x = 1134$ $\frac{I_{xy}}{I_x} I_{xy} = 1$ $I'_y = 361$

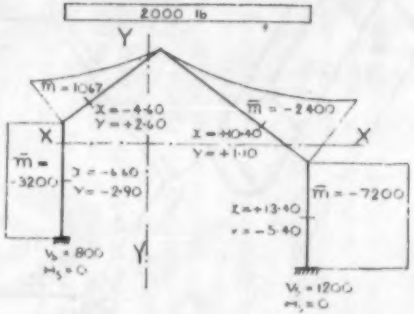


Fig. 5.

TABLE III.

MEMBER	a	$\bar{m}$	$p = \bar{m}a$	x	y	px	py
AB	8·00	−3200	−25600	−6·60	−2·90	+169000	+74300
BC	5·00	−1067	−5330	−4·60	+2·60	+24550	−13860
CD	4·80	−2400	−11550	+10·40	+1·10	+120200	−12710
DE	2·25	−7200	−16210	+13·40	−5·40	+217300	+87500

$P = -58890$ $M_x = +143950$ $M_y = +135230$

POINT	m_2	m_1	$m = m_1 - m_2$
A	−3200	−4680	+1480
B	−3200	−1610	−1590
C	0	− 395	+ 395
D	−7200	−5095	−2105
E	−7200	−8550	+1350

$\frac{M_y}{I_y} I_{xy} = +10650$ $\frac{M_x}{I_x} I_{xy} = -3620$

$M'_x = -154600$ $M'_y = +138850$

$\bar{m}_1 = -\frac{58690}{20{\cdot}05} - \frac{154600}{1134}x + \frac{138850}{361}y$

$= -2930 - 136x + 384y$

$H = 384$, $V_A = 936$, $V_1 = 1064$

rollers or hinges it is made statically determinate, and the moment m_s at any point can be determined. This moment is then applied to the corresponding point of the analogous column as a direct load of intensity m_s per unit of area. The following quantities are then calculated:

$$P = \Sigma(m_s \times a); \quad M_x = \Sigma(m_s \times a \times x); \quad M_y = \Sigma(m_s \times a \times y); \quad A = \Sigma a.$$

The indeterminate moment m_i may now be found for any point on the frame from the expression

$$m_i = \frac{P}{A} + \frac{M'_x}{I'_x}x + \frac{M'_y}{I'_y}y \qquad . \quad . \quad . \quad . \quad (1)$$

where $M'_x = M_x - \frac{I_{xy}}{I_y}M_y$, $M'_y = M_y - \frac{I_{xy}}{I_x}M_x$, $I'_x = I_x - \frac{I_{xy}}{I_y}I_{xy}$,

and $$I'_y = I_y - \frac{I_{xy}}{I_x}I_{xy}.$$

This expression is the general form of the equation for the stress at any point in an unsymmetrical section of a column. The final moment m at any point in the frame is given by

$$m = m_s - m_i \qquad . \quad . \quad . \quad . \quad . \quad (2)$$

The term $\frac{M'_x}{I'_x}$ is the change in the vertical force from that found in the statically determinate condition, and $\frac{M'_y}{I'_y}$ is the change in the horizontal force.

As an example, the frame shown in *Fig.* 3 is analysed. To determine the centroid of the analogous column an origin is taken at A and the calculation is shown in *Table* I. The properties of the analogous column are determined in *Table* II. The terms i_x and i_y are the moments of inertia of each member about its centroid parallel to axes X—X and Y—Y and i_{xy} is the product of inertia about its centroid (*Fig.* 4).

The frame may now be analysed for any condition of loading. Two conditions will be considered. (1) A uniform vertical load of 2000 lb. and (2) A concentrated horizontal load of 100 lb.

Case 1 (*Fig.* 5).—To make it statically determinate the frame is cut at C and the resulting moment-diagram, the value of $\overline{m}$ (the average intensity of the analogous load on each member), and the co-ordinates of the centroid of the load are shown. The values of P, M'_x and M'_y are derived in *Table* III and the forces and final moments at each point are indicated.

Case 2 (*Fig.* 6).—The frame is cut at B, and the calculation is shown in *Table* IV.

In a similar manner any other system of loading may be analysed.

Symmetrical Frames.—The method is similar to the general case except that I_{xy} is zero and the expression for the indeterminate moment reduces to

$$m_i = \frac{P}{A} + \frac{M_x}{I_x}x + \frac{M_y}{I_y}y \qquad . \quad . \quad . \quad . \quad (3)$$

The frame in *Fig.* 7 is analysed for a uniform vertical load on the rafters. The calculations are shown in *Fig.* 8 and *Tables* V, VI, and VII. It should be

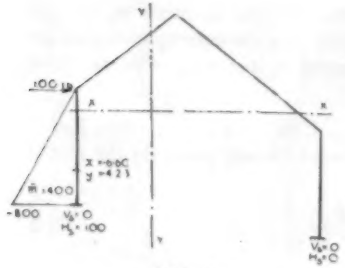


Fig. 6.

TABLE IV.

MEMBER	a	m̄	p = m̄a	x	y	px	py
AB	8·00	-400	-3200	-6·60	-4·23	+ 21100	+ 13550
POINT	m_s	m_i	$m = m_s - m_i$				
A	-8·00	-525	-275				
B	0	-2·37	+2·37				
C	0	+1·21	-1·21				
D	0	+4·5	-4·5				
E	0	-2·80	+2·80				

$\frac{M_x}{I_x} I_{xy} = +1085, \frac{M_y}{I_y} I_{xy} = +530$

$M'_x = +20035 \qquad M'_y = +13020$

$m_i = -\frac{3200}{20·05} + \frac{20035}{1134} x + \frac{13020}{361} y$

$= -160 + 17·65 x + 36·1 y$

$V_A = 18 \quad V_E = 18 \quad H_A = 64 \quad H_E = 36$

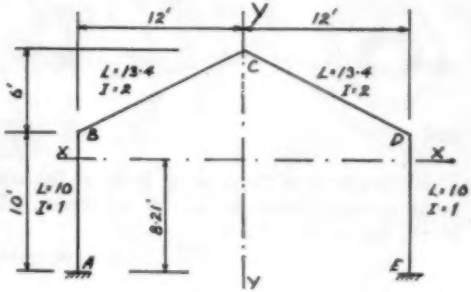


Fig. 7.

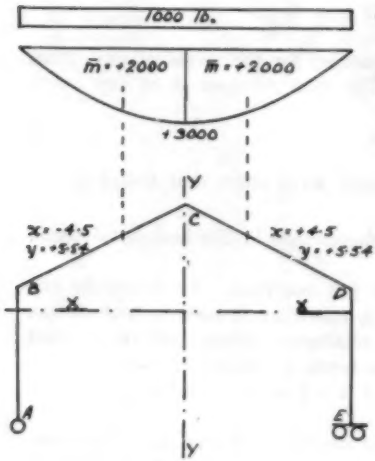


Fig. 8.

TABLE V.

MEMBER	L	$\frac{1}{2}$	a	y_1	ay_1
AB	10·0	1·0	10·0	+5·0	+50·0
BC	13·4	0·5	6·7	+13·0	+87·1
			16·7		137·1

$\bar{y} = 8·21$

TABLE VI.

MEMBER	a	x	y	$ax^2 + i_x$	$ay^2 + i_y$
AB	10·0	-1·20	-5·21	1440	103
				0	83
BC	6·7	-6·0	+4·79	241	154
				80	20
	$\frac{16·7}{2}$			$\frac{1761}{2}$	$\frac{360}{2}$

$\Sigma a = 33·4 \qquad I_x = 3522 \quad I_y = 720$

TABLE VII.

MEMBER	a	m̄	p = m̄a	y	py
BC	6·7	+2000	+13400	+5·54	+74200

$P = \frac{+26800}{2} \qquad M_y = \frac{+148400}{2}$

$m_i = \frac{+26800}{33·4} + \frac{148400}{720} y$

POINT	m_s	m_i	$m = m_s - m_i$
A & E	0	-890	+890
B & D	0	+1171	-1171
C	+3000	+2407	+593

$= 802 + 206 y$

$H = 206 \quad V = 500$

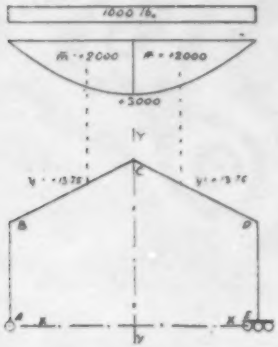


Fig. 9.

TABLE VIII.

MEMBER	L	$\frac{1}{I}$	Q	y	$Qy^2 + I_y$
AB	10·0	1·0	10·0	5·0	250
					83
BC	13·4	0·5	6·7	13·0	1132
					20
					1485
					2
					$\frac{1}{I_y}$ = 2970

TABLE IX.

MEMBER	Q	$\bar{m}$	$P=\bar{m}Q$	Y	PY
BC	6·7	+2000	+13400	13·75	184000
POINT	m_s	m_i	$m=m_s-m_i$		$\frac{1}{I}$
B & D	0	+1240	−1240		M_y = 368200
C	+3000	+1985	+1015		$m_i = \frac{368200}{2970} y$
					= 124 y

H=124 V=500

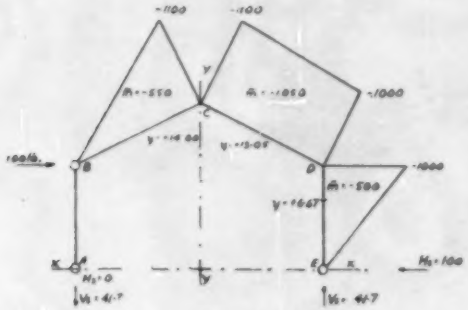


Fig. 10.

TABLE X.

MEMBER	Q	$\bar{m}$	$P=\bar{m}Q$	y	PY
BC	6·7	− 550	−3680	+14·00	−51,500
CD	6·7	−1050	−7030	+13·05	−91,700
DE	10·0	− 500	−5000	+ 6·67	−33300
POINT	m_s	m_i	$m=m_s-m_i$		
A	0	0	0		M_y = −176500
B	0	− 595	+595		$m_i = -\frac{176500}{2970} y$
C	−1100	− 950	−150		= −59·5 y
D	−1100	−595	−405		
E	0	0	0		

V = 41·7

H_A = 59·5 H_E = 40·5

noted that in *Tables* V and VI only half the frame need be considered, and for the particular case of a symmetrical load M_x is zero and need not be entered in *Table* VII.

FRAMES WITH COLUMNS HINGED AT THE BASES.—In these cases the hinges represent points of zero stiffness and hence in the analogous column are represented by infinite areas. The centroid lies midway between the hinges, and the axis X—X passes through them. The area A and I_x are infinite, and it is therefore

only necessary to calculate I_y and M_x. The frame of *Fig.* 7 is shown in *Fig.* 9 hinged at the bases, and the calculations are shown in *Tables* VIII and IX.

It should be noted that the frame must be made statically determinate by the introduction of another hinge. The frame cannot be made rigid at any point in order to use a cantilever moment-diagram. In *Fig.* 10 and *Table* X the frame of *Fig.* 9 is analysed for a concentrated horizontal load.

FRAMES WITH ONE COLUMN HINGED AT THE BASE.—In these cases the elastic centre is at the hinge and the analogous area is infinite. The frame is dealt with as in the general case for an unsymmetrical frame, and the first term in the expression for m_i in equation (1) becomes zero.

THE LATE OSCAR FABER, C.B.E., D.Sc.
(*See page* 387.)

Bridges Liable to Mining Subsidence.

MINING has been known to cause subsidence exceeding 10 ft. over a period of years, although in the main it occurs gradually. The extraction of brine often has, however, more sudden and sometimes greater effects. The following method (see *Fig.* 1) has been suggested by Mr. Donovan Lee, M.I.C.E., for dealing with subsidence due to any form of mining, and to some causes other than mining that may result in settlement.

Essentially the principle employed is that a three-point support enables any one support to suffer relative vertical movement without altering the stresses in the supported structure. In Mr. Lee's proposal this principle is extended to apply to both the foundations and to the girders and deck of a bridge, so that the span is supported on the abutments or piers at two points at one end of the span but at only one point at the other end, as shown in the sketch. This does not prevent, but rather facilitates, provision in the abutments or piers for jacking up the deck when desired at points immediately adjacent to one or more of the three supports.

Owing to the fact that subsidence over a period may result in the abutments or piers being slightly out of the vertical, the foundations are preferably arranged so that the load is within the triangle formed by the three points of support. It is, of course, possible to arrange the foundations the reverse way to that shown in the diagram so that the third support is under the span instead of under the embankment. The principles are not altered if piled foundations are needed in place of spread foundations, but in this case the piles are preferably arranged as close as practical to the points of support. Due to the avoidance of stresses caused by the warping of normal structures as a result of subsidence, and the difficulty and cost of providing for it, this method is expected to be more economical than methods now in use.

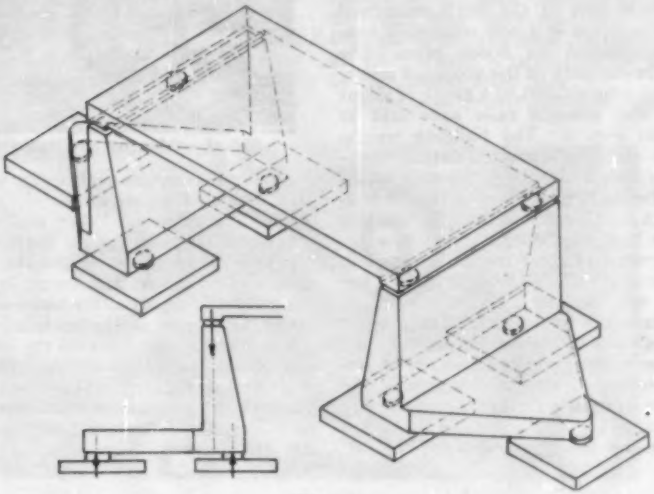


Fig. 1.

Moving an Ancient Monument.

DURING the reconstruction of a war-damaged part of Marseille the new site of a main road threatened the destruction of the Hôtel de l'Echevin de Cabre built in the year 1535. This ancient building was preserved by the efforts of the Service des Monuments Historiques who proposed moving it a distance of about 80 ft. and through an angle of 80 deg. (*Fig.* 1). The work is described in an article by M. K. Olsen and M. J. Prevost in a recent number of the Belgian journal "La Technique de Travaux".

The first part of the work comprised the construction of a new rein.forced concrete foundation on bored piles. The ground in the path of the proposed movement was consolidated to a depth of about 10 ft., and straight rails were laid as shown in *Fig.* 2. The building weighs 700 tons and was supported during movement on four hydraulic jacks each on an eight-wheeled bogie. The jacks were connected in such a manner that the building could be kept level during the movement. A framework of steel beams, inserted in the masonry walls about 5 ft. above the ground, was used to transmit the weight of the structure to the jacks, but in order to reduce the size of these beams the walls were also prestressed horizontally. Tie-bars were placed at intervals around the structure to reduce further the possibility of cracking.

Fig. 1.—Moving the Building.

The bogies were pushed along the rail by hydraulic devices anchored at intervals to the rails, the rate of movement being 4½ in. per minute. When the building was in the new position the space between the base of the structure and the foundation was concreted and some days later the weight of the structure was transferred from the bogies to the foundation. No damage had been done to the structure during the entire operation. The work was carried out by Établissements Christiani et Nielsen under the direction of M. Colas, Chief Architect of the Service des Monuments Historiques.

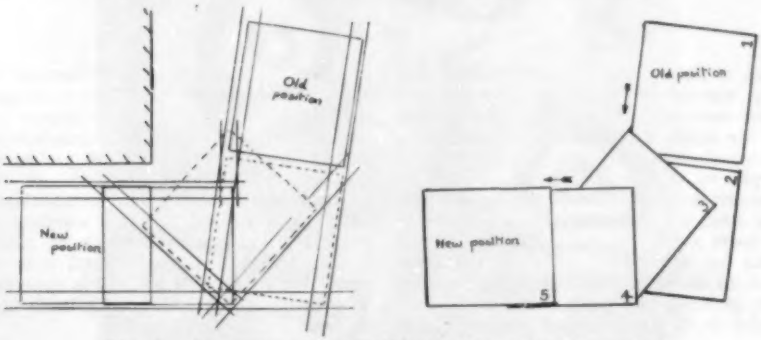


Fig. 2.—Arrangement of Rails and Stages of Movement.

A Stand in South Africa.

A NEW stand (*Figs.* 1 and 2), with a seating capacity of 2000, has been constructed at the Free State Stadium, Bloemfontein, South Africa. The total length of the stand is 188 ft., and it is proposed to increase this by 376 ft. later. The stand comprises seven reinforced concrete cantilevered frames at 28 ft. 6 in. centres, with six intermediate frames below the seats. The roof is of reinforced concrete arches 2¼ in. thick with ribs spanning between the cantilevers. The front and rear columns supporting the frames are 18 in. wide and taper from top to bottom. They are all hinged at their bases. The cantilevered frames span 55 ft. 6 in. Between the cantilevered and the intermediate frames there are twenty rows of precast seats 13 ft. 9 in. long, grouted together at the joints. Below the stand are dressing rooms, an assembly hall, a boiler room, and other accommodation. The middle two sections at the top rear of the stand are enclosed to accommodate the Press and radio commentators.

The roof slabs and ribs have a rise of 2 ft. at the centre of the span and were designed for an imposed load of 10 lb. per square foot. Between adjacent cantilevered beams are seven arch ribs and, except for the front two and the one at the rear, they have tie-beams horizontally above them to resist the thrust of the arch. The roof is reinforced by a mesh of high-tensile steel bars.

In the cantilevered frames the maximum

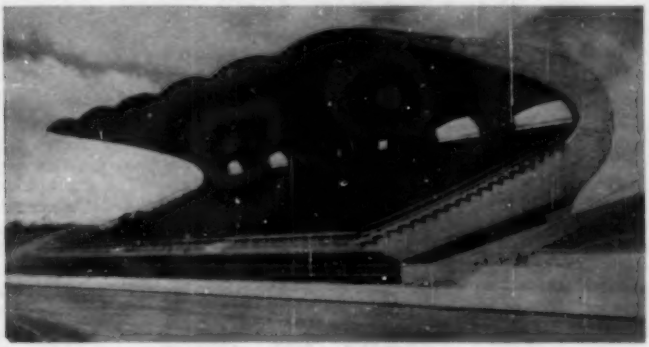


Fig. 1.

Fig. 2.

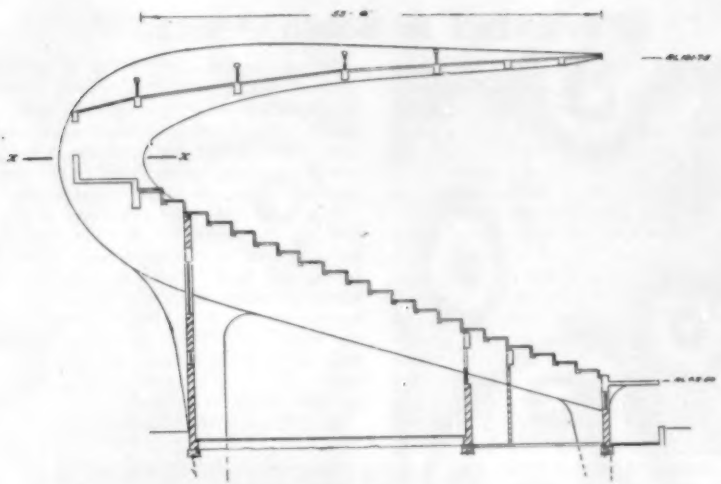


Fig. 3.

bending moment occurs at section X–X (*Fig.* 3). The dimensions of the beam at this section are 10 ft. by 2 ft. and the tensile reinforcement comprises twenty-six 1¼-in. bars. From a point just above section X–X the cantilevered beams taper in plan from 2 ft. to 1 ft. at the extreme end. Nominal reinforcement to resist shearing forces was provided in the cantilevered beams.

The lower portions of the cantilevered frames are 2 ft. thick and were cast in one lift from the front column to a point just below section X–X. Thirty-five cubic yards of concrete were required. After a minimum period of fourteen days the remaining sections of the cantilevers were cast, with rebates and splice bars provided at the junction of the slabs and ribs. The slabs and ribs were then cast between two adjacent cantilevered beams, screeded to shape, and smoothed with a wooden float. Later the slab was rubbed with cement-and-sand mortar to fill in any holes and cracks. No expansion joints are provided in the roof. Steel shutters were used for the frames and deck. The soffits of the cantilevered beams were supported from the lower beams on 9-in. by 3-in. timber props. The concrete mixture was 1 : 1½ : 3, and was vibrated. The strength at 28 days varied from 4500 lb. to 7000 lb. per square inch. The minimum strength required was 3500 lb. per square inch. The average deflection of the ends of the cantilevered beams when the props were removed was 1¼ in.

The rear columns are not at the point of balance of the structure, but are a few feet farther back to provide additional space under the stand. This resulted in a bending moment due to the live and dead loads of the roof and cantilevered beam of about 1,060,000 ft.-lb. to be distributed between the column and the lower half of the cantilevered frame.

Access to the stand is by a cantilevered walkway 6 ft. wide in front, with two cantilevered staircases 7 ft. wide at each end. There is also a staircase from the entrance hall under the stand to an enclosure at the centre. Below the cantilevered walkway at the front of the stand there is a gallery with seating for 80 competitors.

The consulting engineer was Mr. Ninham Shand, B.Sc., M.I.C.E. (Bloemfontein branch), and the contractors were du Plessis-Lewis Construction (Pty.), Ltd., Bloemfontein.—R. I. THERON.

Design of Tee-beams and Ell-beams.

By R. J. BARTLETT, A.M.I.Struct.E.

TEE and ell beams may be designed or analysed rapidly and fairly accurately by substituting in the following formulæ the values of P, K, L, and M read from *Fig.* 1. These values depend on the ratios of the depth of the slab to the effective depth of the beam.

The percentage of steel r is $\frac{10Q(5+K)}{t} - \frac{M}{100}$ . . . (1)

The maximum stress in the concrete c is $PQ + \frac{tL}{100}$ . . . (2)

A tee or ell beam will act as a normal rectangular beam if Q is less than $\frac{2tV}{1000}$, where V is read from the curve in *Fig.* 1.

The method may be employed to design rectangular beams by equating $\frac{2tV}{1000}$ to Q in order to obtain V. The values of P, K, L, and M corresponding to V may be read from the graphs and substituted in equations (1) and (2).

EXAMPLE I.—A tee-beam 4 in. thick, with a flange 56 in. wide and an effective depth of 16·5 in., is subjected to a bending moment of 1,500,000 in.-lb. If the tensile stress in the reinforcement is 18,000 lb. per square inch, calculate the maximum stress in the concrete and the amount of reinforcement required.

$$s = \frac{4}{16{\cdot}5} = 0{\cdot}2425. \quad Q = \frac{1{,}500{,}000}{56 \times 16{\cdot}5^2} = 98{\cdot}5.$$

From the graph, $P = 5{\cdot}32$, $K = 6{\cdot}31$, $L = 0{\cdot}87$, and $M = 1{\cdot}02$, and from formulæ (1) and (2)

$$r = \frac{10 \times 98{\cdot}5(5 + 6{\cdot}31)}{18{,}000} - \frac{1{\cdot}03}{100} = 0{\cdot}61 \text{ per cent.}$$

$$c = 5{\cdot}32 \times 98{\cdot}5 + \frac{18{,}000 \times 0{\cdot}88}{100} = 682 \text{ lb. per square inch.}$$

EXAMPLE II.—As Example I but with a slab 6 in. thick. $s = \frac{6}{16{\cdot}5} = 0{\cdot}364$. $Q = 98{\cdot}5$ as before. $\frac{2tV}{1000} = \frac{2 \times 18{,}000 \times 3{\cdot}07}{1000} = 110{\cdot}5$.

Since Q is less than 110·5 the beam acts as a rectangular beam. Therefore $\frac{2tV}{1000} = Q = 98{\cdot}5$, that is $V = 2{\cdot}74$. From the graphs, $P = 4{\cdot}16$, $K = 6{\cdot}93$, $L = 1{\cdot}29$, $M = 3{\cdot}35$.

$$r = \frac{10 \times 98{\cdot}5(5 + 6{\cdot}93)}{18{,}000} - \frac{3{\cdot}35}{100} = 0{\cdot}62 \text{ per cent.}$$

$$c = 4{\cdot}16 \times 98{\cdot}5 + \frac{18{,}000 \times 1{\cdot}29}{100} = 642 \text{ lb. per square inch.}$$

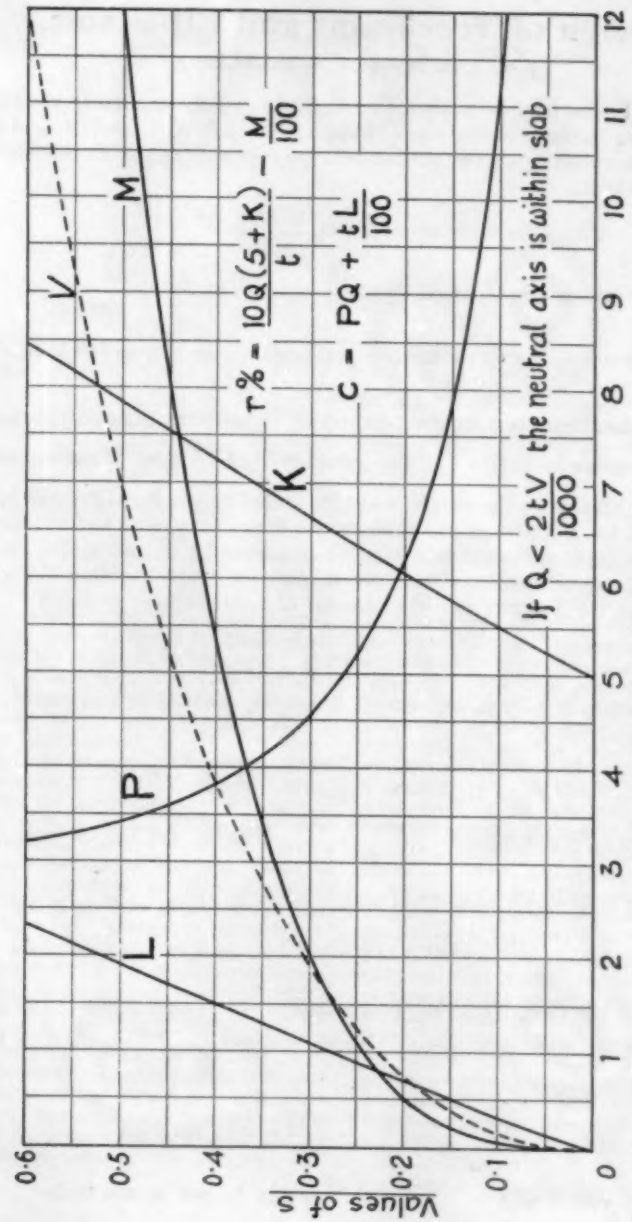


Fig. 1.—Design Constants for Tee-beams and Ell-beams.

EXAMPLE III.—As Example I but, if the maximum stress in the concrete is 1000 lb. per square inch and the flange is 30 in. wide, calculate the moment of resistance of the beam and the necessary amount of reinforcement. P, K, L, and M are as in Example I.

Using formula (2), $1000 = 5{\cdot}32 \times Q + \dfrac{18{,}000 \times 0{\cdot}88}{100}$.

Therefore $$Q = \frac{1000 - 158{\cdot}3}{5{\cdot}32} = 158,$$

and the moment of resistance is $158 \times 30 \times 16{\cdot}5^2 = 1{,}290{,}000$ in.-lb.

$$r = \frac{10 \times 15{\cdot}8(5 + 6{\cdot}31)}{18{,}000} - \frac{1{\cdot}02}{100} = 0{\cdot}985 \text{ per cent.}$$

Method of Deriving the Graph.

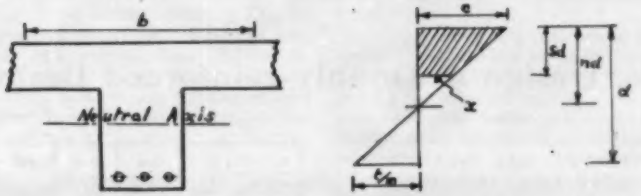


Fig. 2.—Stress-Diagram for a Tee-beam.

Using the notation in *Fig.* 2, from the stress-diagram

$$\frac{c}{x} = \frac{nd}{nd - sd} \text{ and } \frac{cm}{t} = \frac{nd}{d - nd}.$$

Therefore $x = c - \left(\dfrac{t}{m} + c\right)s$.

Since the moment of resistance of a beam is equal to the moment of the stress in the concrete about the centroid of the reinforcement,

$$Qbd^2 = x \times sd \times b\left(d - \frac{sd}{2}\right) + \frac{(c - x)}{2}sd \times b\left(d - \frac{sd}{3}\right).$$

Also, the total compression equals the total tension. Therefore

$$\frac{(c + x)}{2}sd \times b = \frac{rbd}{100} \times t.$$

By substituting $c - \left(\dfrac{t}{m} + c\right)s$ for x in these two expressions and rearranging,

$$c = \frac{3Q}{s(3 - 3s + s^2)} + \frac{ts(3 - 2s)}{2m(3 - 3s + s^2)} \quad . \quad . \quad . \quad (3)$$

$$r = \frac{3(2 - s)Q}{2t(3 - 3s + s^2)} - \frac{s^3}{4m(3 - 3s + s^2)} \quad . \quad . \quad . \quad (4)$$

The coefficients of Q and t in equation (3), and $\frac{Q}{t}$ and the last term in equation (4), are functions of s which may be denoted by P, $\frac{L}{100}$, $10(5 + K)$, and $\frac{M}{100}$ respectively. Then, by substitution, formulæ (1) and (2) follow.

From the foregoing it may be proved that

$$n = \frac{6mQ + ts^2(3 - 2s)}{6mQ + 3ts(2 - s)} \quad . \quad . \quad . \quad . \quad (5)$$

If the neutral axis is within the flange, n must be less than s, and by rearranging (5), Q must be less than $\frac{ts^2(3 - s)}{6m(1 - s)}$, that is, Q must be less than $\frac{2tV}{1000}$, where V is a function of s. P, K, L, M and V are represented graphically in *Fig.* 1 with m assumed to be 15.

The Design of Doubly-reinforced Beams.

MR. J. S. SAVONA writes as follows.

In an article by the writer in this journal for November, 1954, the solution of the quadratic equation for n_1 is given as

$$n_1 = \frac{-A + \sqrt{A^2 + B}}{2} \quad . \quad (2b)$$

where $$\frac{A}{p} = 2[\phi(m - 1) + m] \quad . \quad (3a)$$

and $$\frac{B}{p} = 8[\phi k_1(m - 1) + m] \quad . \quad (4a)$$

A quicker method is to replace $\frac{B}{p}$ by a new factor

$$\frac{\omega}{p} = 2[\phi k_1(m - 1) + m] \quad . \quad (4b)$$

The quadratic equation for n_1 becomes

$$n_1^2 + An_1 - \omega = 0 \quad . \quad (2c)$$

Values of $\frac{A}{p}$ and $\frac{\omega}{p}$ for various values of m and ϕ are shown in *Table* I. The equation can be solved quickly by a slide-rule with a Cr scale, or by inverting the slide and reversing it lengthwise. Set the index of scale C over ω on scale D. Move the cursor to a position such that the reading on scale Cr (or the inverted scale C) less the reading on scale D is equal to A. The reading on scale D is the value of n_1.

TABLE I.

$K_r = 0.05$	ϕ =	0·6	0·8	1·0	1·2
VALUES OF m: 8	A/p	24·40	27·20	30·00	32·80
	ω/p	16·42	16·56	16·70	16·84
10	A/p	30·80	34·40	38·00	41·60
	ω/p	20·54	20·72	20·90	21·08
12	A/p	37·20	41·60	46·00	50·40
	ω/p	24·66	24·88	25·10	25·32
15	A/p	46·80	52·40	58·00	63·60
	ω/p	30·84	31·12	31·40	31·68
18	A/p	56·40	63·20	70·00	76·80
	ω/p	37·02	37·36	37·70	38·04
20	A/p	62·80	70·40	78·00	85·60
	ω/p	41·14	41·52	41·90	42·28
22	A/p	69·20	77·60	86·00	94·40
	ω/p	45·26	45·68	46·00	46·52

EXAMPLE.—In example I in the article referred to $p = 0.01$.

$\frac{A}{p} = 52.4$; therefore $A = 0.524$.

$\frac{\omega}{p} = 31.12$; therefore $\omega = 0.3112$.

Place the index of scale C over 0·3112 on scale D and, by moving the cursor, obtain readings on Cr and on D such that their difference is equal to $A = 0.524$. The reading on scale D is n_1 and is found to be 0·354. Having determined n_1, the design can be completed.

Design for a Bridge over the River Tiber.

On page 211 of the January 1956 number of this journal a photograph was given of a model of a design for a bridge submitted in a competition organised by the City of Rome for a new bridge to replace the existing Garibaldi bridge over the river Tiber. Professor Carlo Cestelli-Guidi has now sent us the accompanying illustrations and the following notes on this unusual design.

The competition called for a bridge to be built on the two abutments and the central pier, rather more than 10 m. (32·8 ft.) wide, of the existing two-arched iron bridge. Also, the bridge would have to be constructed one half at a time. Photographs of a model are shown in *Figs.* 1 and 2, and the arrangement in *Fig.* 3. The supporting structure consists of two reinforced concrete arches of 40 m. (131·2 ft.) span. At the abutments the arches rest on piles of large diameter and the horizontal thrust is transmitted to the existing abutments as shown in *Fig.* 3; the other end is supported on an inverted arch supported by the masonry of the existing pier.

The arches have a cylindrical profile and their thickness varies axially and transversely so that, while they give the impression of a pair of semi-circular arches, thus recalling a characteristic of Roman bridges, they have an almost straight central longitudinal strip corresponding with the line of thrust of the load concentrated at the crown of the arch. An advantage of this form of construction is that the live load produces variations of force, but only small bending moments, in an axial direction. The arches were designed as two-pinned structures by the exact method of the ellipse of elasticity. In fact, because of the great reduction of the thickness at the centre of the span, the behaviour of the structure would be very like that of a three-pinned arch. Rocker bearings (*Fig.* 4) would be provided at the crowns of the arches.

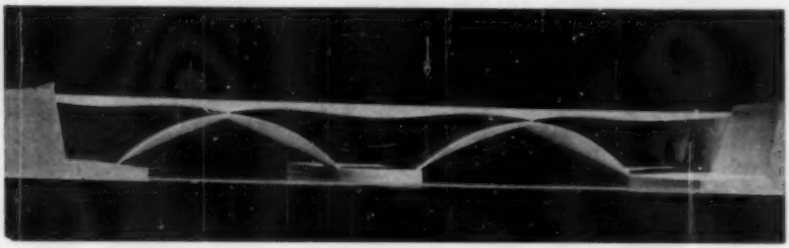


Fig. 1.

Fig. 2.

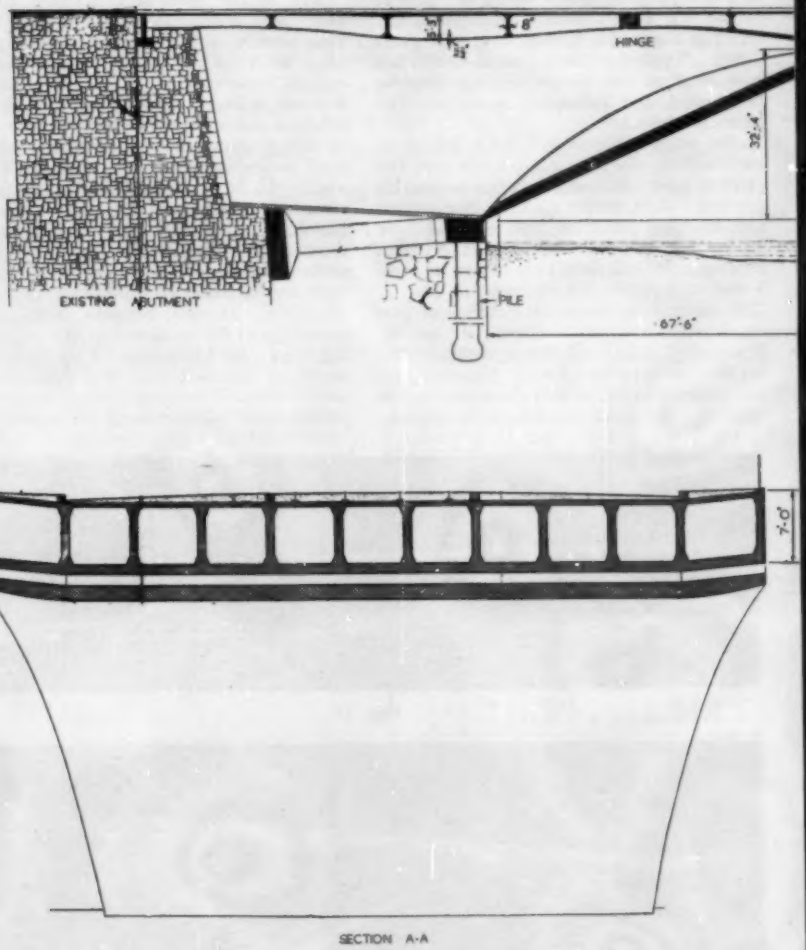
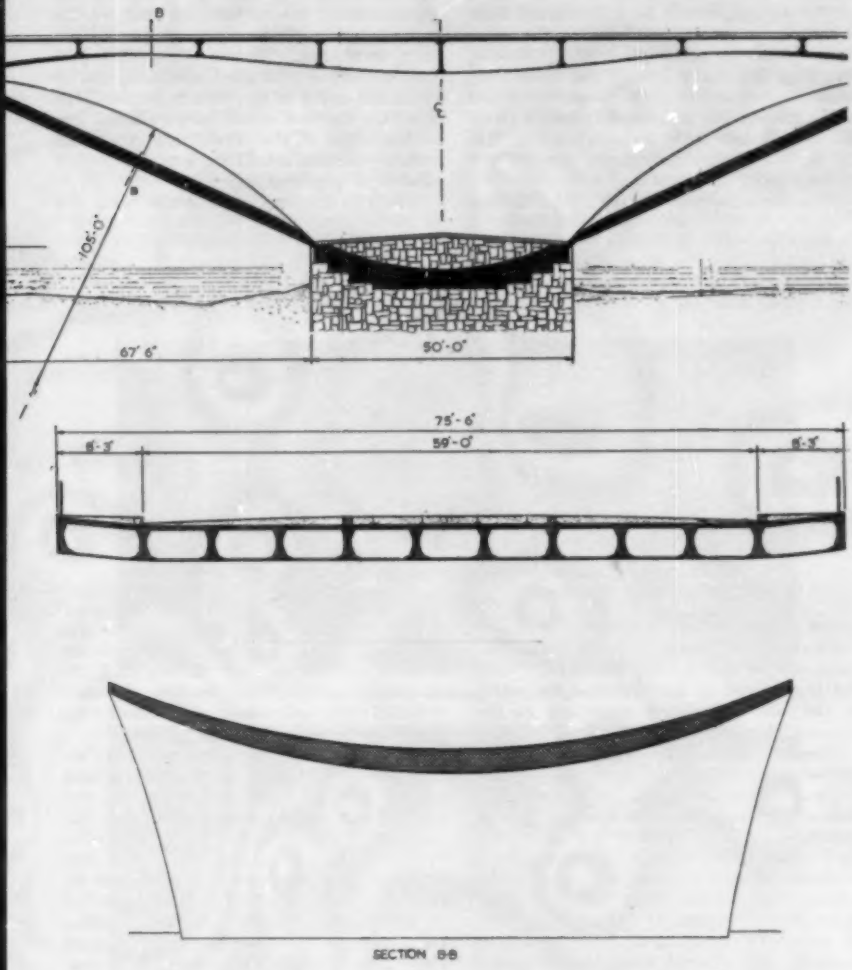


Fig. 3.—General Arrangemen



idge shown in Figs. 1 and 2.

The arches would be prestressed transversely by cables comprising twelve 5-mm. (0·197-in.) diameter wires in order to provide a small compression in a transverse direction, particularly as one half of the bridge would be constructed first. For this reason it was intended that one-half should be prestressed first, followed later by the other half. The maximum axial stresses in the arches were calculated to be 72 kg. per square centimetre (1025 lb. per square inch) in compression and 14 kg. per square centimetre (200 lb. per square inch) in tension.

The deck would consist of a hollow girder construction with twelve longitudinal ribs of depths varying with the incidence of the bending moment. Also, the thicknesses of the slab and the width of the ribs are varied according to the shearing forces.

The longitudinal beams would be prestressed with cables containing twelve 7-mm. (0·276-in.) diameter wires. The slab and transverse beams would not be prestressed. This system has been used by Professor Carlo Cestelli-Guidi in other constructions with excellent results. The longitudinal beams would have a hinge in each of the two end-spans and thus be statically determinate. These hinges would be fixed when shrinkage and any other deformation had taken place so that the beams would act as a continuous statically-indeterminate structure.

The maximum stresses in the beams under full load were calculated to be 140 kg. per square centimetre (2000 lb. per square inch) in compression and 14 kg. per square centimetre (200 lb. per square inch) in tension. It was intended to provide sufficient mild steel reinforcement to resist the whole of the tensile forces. The shearing stresses would be very small due to the slope of the cables, and the main tensile stresses would be low due to the effects of prestressing.

The calculations were made with the use of influence lines for the stresses due to live loads, considering the elastic system formed by the arches and the statically-indeterminate beams. Military vehicles with axle loads of 19 short tons were assumed to form the live load.

The design was made by Professor Cestelli-Guidi, of the University of Rome, and the architect Sr. Bruno Zevi in collaboration with Professor Gentiloni Silvery and the architects Srs. A. Di Carlo, J. Ferris, and M. Goldsmith. The construction methods were worked out by Sr. Carlo Allegri. The project will not be carried out because a more economical scheme for restoring and widening the old bridge is considered to be preferable.

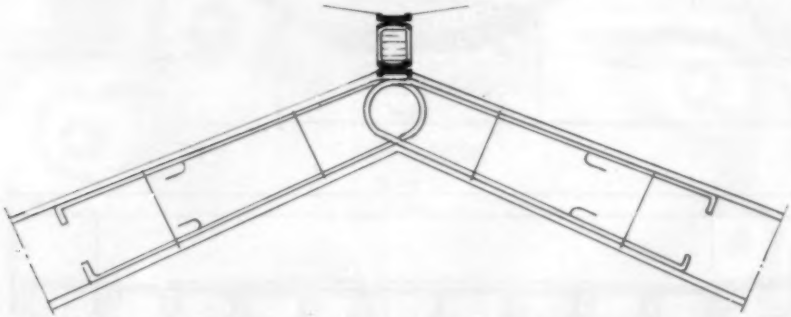


Fig. 4.—Rocker Support at Crown of Arch.

A Tank for Hydro-dynamic Research.

An annular tank, with a capacity of more than a million gallons, for the investigation of the behaviour of bodies moving at high speed under water has recently been completed at the Admiralty Research Laboratory at Teddington. Supported from a rotating beam, models of up to 20 ft. long can be tested, at a depth of 5 ft., at speeds up to 100 knots. The water will be treated to permit the use of high-speed photography, and objects will be clearly visible in the water at distances up to 100 ft. Observation is from two underwater chambers with glass windows.

The tank (*Figs.* 1 and 2) is of reinforced concrete and is 136 ft. maximum diameter by 34 ft. wide by 15 ft. deep, with a tunnel below. The excavation through water-bearing ballast into blue clay was facilitated by dewatering the ground, and the sheet piles used to keep water from the excavation were used as shutters for the outer wall of the tank. The annular walls were designed to withstand a head of water of 15 ft. At the bottom the inner wall is 1 ft. 6 in. thick and the outer wall 1 ft. thick, backed by 1 : 20 plain concrete 5 ft. 4½ in. thick by 5 ft. deep. The floor has a least thickness of 2 ft. and was designed to resist an external head of water of 11 ft. 4 in.

The floor was cast in sections not exceeding 25 ft. by 25 ft. with gaps 1 ft.

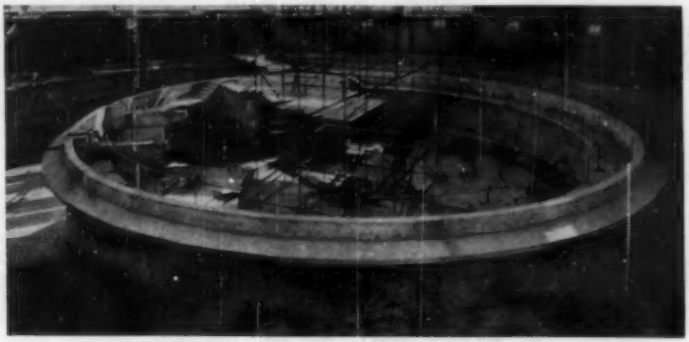


Fig. 1.—Tank during Construction.

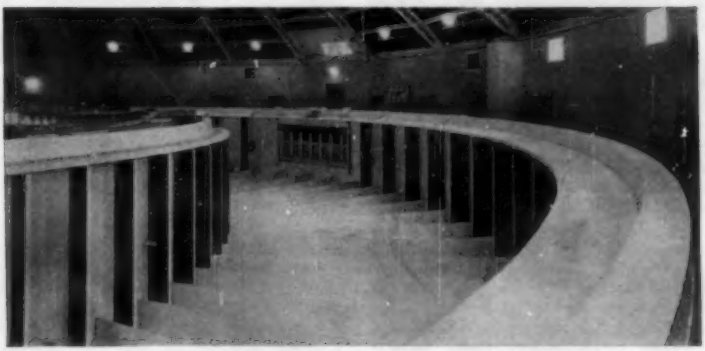


Fig. 2.—Part of Completed Tank, showing Projections to Prevent the Water Swirling.

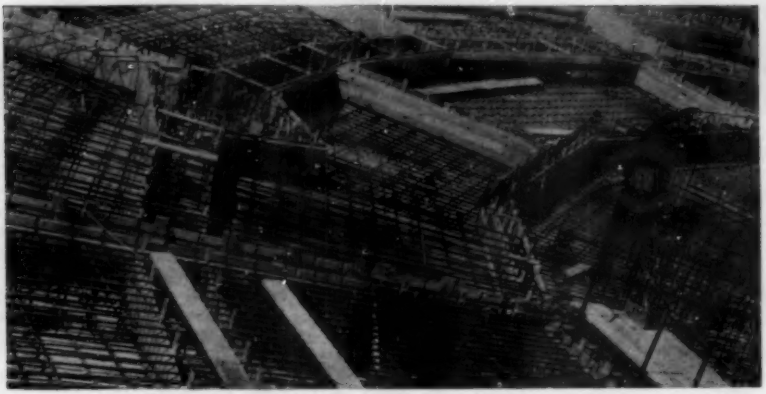


Fig. 3.—Floor during Construction.

wide radially and circumferentially (*Fig.* 3). The walls were cast in lengths not exceeding 30 ft. with gaps 2 ft. wide and in lifts not exceeding 3 ft.; the gaps were filled after shrinkage had taken place in the concrete already placed. As soon as possible after casting the shuttering was removed and the edges at the gaps were subjected to a jet of water to expose the aggregate. Immediately before concreting the gaps the surfaces were cleaned by a jet of air, and a layer of concrete was placed and vibrated against the dry surface. No water-bars or rebates were used in the joints. It is claimed that a high degree of watertightness was achieved; the few places where there was slight seepage were made good by pressure grouting.

All the concrete was proportioned by weight and vibrated. In the work below ground and in the floor the concrete comprised 112 lb. of Portland cement, 210 lb. of aggregate of $\frac{1}{16}$-in. maximum size and 300 lb. of aggregate $1\frac{1}{4}$ in. to $\frac{3}{4}$ in. The concrete for the walls comprised 112 lb. of Portland cement, 179 lb. of sand of $\frac{1}{16}$-in. maximum size, and 358 lb. of aggregate $\frac{3}{4}$ in. to $\frac{3}{8}$ in. In both cases the water-cement ratio was 0·42 to 0·44 and the compressive strength at 28 days was specified to be not less than 4000 lb. per square inch. Test cubes showed that the first mixture varied in strength from 4930 lb. to 3320 lb. per square inch and the second mixture from 5650 to 4170 lb. per square inch.

The shutters were of timber and held together by wire ties passing through the walls. The shutters were stripped early and minor blemishes made good with a 1 : $1\frac{1}{2}$ mixture of cement and sand. The concrete was rubbed down with fine carborundum before it had hardened, and surfaces exposed to water were painted with chlorinated rubber paint.

The tank was designed for the Admiralty by the Chief Structural Engineer's Division, in conjunction with the Chief Architect's Division, of the Ministry of Works. The main contractors were Messrs. Richard Costain, Ltd.

Road Work in Staffordshire.

WORK has started on the improvement of the Winchester–Preston road (A34) between Stafford borough boundary and Wood Farm, Aston, at a cost of £230,000. The existing carriageway is 22 ft. to 24 ft. wide, and the improvement will comprise a second 24-ft. carriageway with a continuous footpath. The scheme was prepared by the Staffordshire County Council on behalf of the Ministry of Transport and Civil Aviation. The contractors are Tarmac, Ltd.

Expanded-shale Aggregate in Prestressed Concrete.

Properties of Expanded Shale.

EXPANDED-SHALE aggregate is now being used in prestressed concrete in the U.S.A. This type of aggregate is produced by heating shale to incipient fusion, when the gases formed in the material expand and produce numberless small cells, which are still present when the material is cooled. Each cell is surrounded with a hard membrane which is highly resistant to the penetration of moisture, and the cellular nature of the material is responsible for its resistance to the transmission of sound and of heat and cold. It is estimated that in the year 1954 the production of expanded-shale aggregate in the U.S.A. and in two provinces of Canada was about 2,250,000 cu. yd. The capital cost of a plant is about 1000 dollars (£360) per cubic yard of daily capacity.

When it is cool the material is crushed to the required size. The dry loose material has a density of about 55 lb. per cubic foot. The chemical analysis of an aggregate of this type is: silica (SiO_2), 59·4 per cent.; iron oxide (Fe_2O_3), 7·71 per cent.; alumina (Al_2O_3), 24·69 per cent.; calcium oxide (CaO), 0·4 per cent.; magnesium oxide (MgO), 2·28 per cent.; alkali (Na_2O), 0·79 per cent.; sulphur dioxide (SO_2), 0·6 per cent.; potassium oxide (K_2O), 2·23 per cent.; loss on ignition, 1·9 per cent. The material is delivered to customers in a moistened condition.

The density of concrete made with expanded-shale aggregate is about 85 lb. per cubic foot. The compressive strength generally used is between 3000 lb. and 4500 lb. per square inch. For most structural purposes the maximum size of the aggregate is ¾ in., and the proportions by volume are generally 40 to 52 per cent. of coarse material combined with 60 to 48 per cent. of material with a maximum size of ¼ in. Due to the angular shape of the particles it is recommended that an air-entraining agent in the proportion of about ½ oz. per 1 cwt. of cement be used in order to increase the plasticity of the mixture and so permit the use of less mixing water. A mixture of 450 lb. of cement to a cubic yard of aggregate produces a compressive strength of about 3000 lb. per square inch at 28 days.

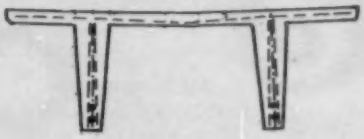


Fig. 1.

Prestressed precast slabs of the double-tee shape shown in *Fig.* 1 are made in spans of up to 50 ft. They are made by the pre-tensioned and the post-tensioned processes. For prestressed work it is common to use this type of concrete with a compressive strength of 3500 lb. per square inch at the time the prestressing force is applied and 4500 lb. per square inch at 28 days. It is claimed that this type of concrete can be made with a compressive strength of 7000 lb. per square inch at 28 days; such concrete weighs about 105 lb. per cubic foot.

The modulus of elasticity of concrete made with expanded shale is about half that of dense concrete, and the loss of prestress due to shrinkage is reckoned to be 25 per cent. of the force applied. This is allowed for in calculating the prestressing force required. Recent tests, however, indicate that an allowance of 15 per cent. is sufficient for loss of prestress.

Tests of Beams.

Tests of beams of the shape shown in *Fig.* 1 have been made at the works at Napa, California, of the Basalt Rock Company. The concrete had a compressive strength of 3500 lb. per square inch when the prestressing force was applied, and 4500 lb. per square inch when the tests were made. They were prestressed with 7-strand wires with a breaking strength in tension of 240,000 lb. per square inch, and these were pre-tensioned to 170,000 lb. per square inch. A double-tee slab 40 ft. long was loaded to three times the load for which it was designed, when the deflection was 1 ft. at the middle and cracks

appeared; when the load was removed the deflection was reduced to 4½ in. When subjected to the load for which it was designed the deflection was not more than two-thirds of the deflection permitted by the code of practice, and no cracks appeared until the load was at least 70 per cent. more than the design load.

Casting by Extrusion.

The method of casting double-tee slabs at the works at Wisconsin of the West Allis Concrete Products Co. is shown in *Figs.* 2 and 3. This is an extrusion process, and produces beams at the rate of 4 ft. 6 in. per minute. A gantry crane (*Fig.* 2) traverses the length of the prestressing beds, and from the gantry is suspended a casting machine which can be arranged to place concrete on any one of six parallel casting beds.

The casting beds are made of reinforced concrete. They are 430 ft. long, and the slabs are made in widths of 20 in. and 40 in. and in thicknesses of 4 in., 6 in., and 8 in. The side forms are at the bottom of the casting machine, and move with it. The concrete is of a very stiff consistency and is consolidated by vibration and tamping so that it retains its shape as the casting machine and side forms move along the bed. The beams are prestressed, and the machine also positions the prestressing wires at the required distance from the bottom of the slab, and smooths the top surface.

When hollow beams are made the machine supplies two classes of concrete, namely a dense concrete for the top and bottom of the slabs and a lightweight concrete around the hollow cores. This is achieved by having two hoppers supplying dense concrete and one hopper supplying lightweight concrete, and the three mixtures are discharged simultaneously. The hollow cores are formed by the machine

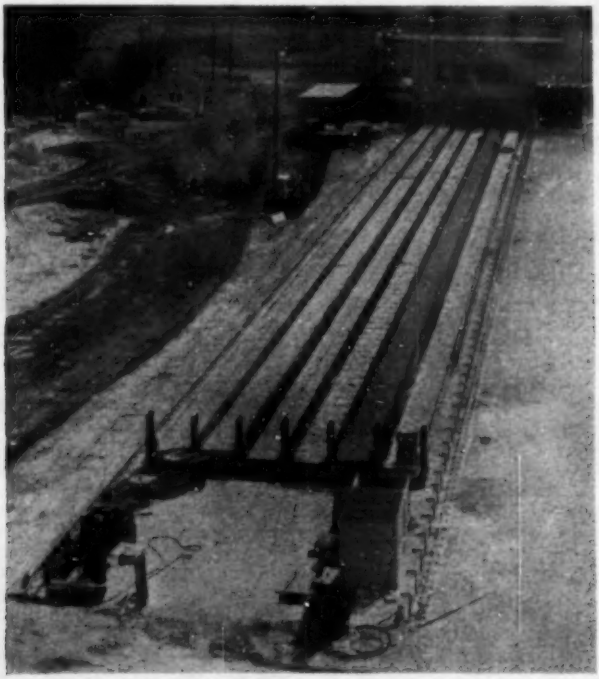
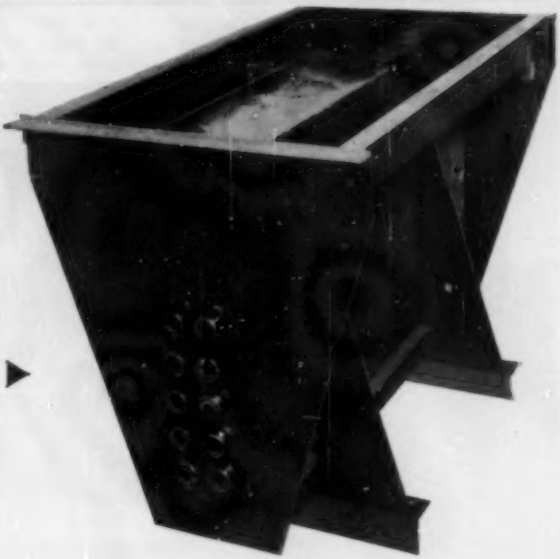


Fig. 2.—The Casting Beds.

We illustrate one of a set of steel moulds that we designed and supplied to the Devon County Council. From this one set of moulds the beams were produced for Lynmouth High Street Bridge, Barbrook Bridge, and Hillsford Bridge.



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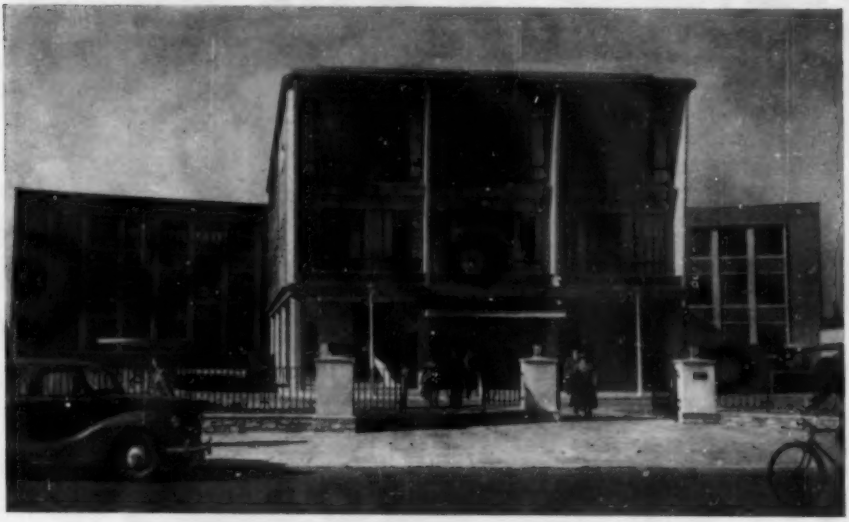

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as it traverses the casting bed. The prestressing is produced by two wires of 0·12 in. diameter twisted together, and they are tensioned to 140,000 lb. per square inch. Some ¼-in. mild steel is used in the tops of the slabs to resist stresses during handling. A hollow slab the full length of the casting bed (430 ft.) is cast in one hour and twenty minutes. At this rate six beds are filled in an 8-hours' day. On the following day more beams are cast on those cast the previous day, and so on until a height of 5 ft. is reached (*Fig.* 3). Steam is circulated through the casting bed, and the concrete has a compressive strength of 4000 lb. per square inch at 28 days. After six days' curing, the wires are cut at the anchorages, and the slabs cut to the required lengths by means of a masonry saw.

One of the longest beams so far made with expanded-shale aggregate in the U.S.A. is 52 ft. long, 4 ft. wide, and 1 ft. 8 in. deep, and has a span of 50 ft. The beam, which was made by the Carter-Waters Corporation, forms a footbridge at Prairie Village, Kansas. There are three vertical stems beneath the slab, each of which is prestressed with 24 high-tensile wires of ¼-in. diameter; the wires were tensioned after the concrete had hardened. Tests showed that the concrete had a strength of 4000 lb. per square inch at two days, 4750 lb. at fourteen days, and 7200 lb. at 28 days.

The production of expanded shale lightweight aggregate is to be started in the autumn of this year by the Butterley Co., Ltd. The works will be at Derby, and will cost £150,000. The raw material will be the shale which is a waste product of coal mining, and it is proposed later to build other works in mining areas where suitable shale is available. The product, which is to be known as "Aglite", is similar to the expanded shale aggregate described in the foregoing.

Fig. 3.—The Concrete Placing Machine.

The Use of Designatory Initials.

In "The Builder" for April 13, 1956, a case is reported in which two Polish "design consultants" were summoned by the Architects' Registration Council because they used the abbreviations Dipl. Ing. Arch. after their names on their notepaper; this was claimed to be contrary to the Architects' Registration Act. The abbreviations denoted a diploma Ingenieur Architekt awarded to them after studying at the Polish School of Architecture attached to Liverpool University for some years after the war. They were each ordered by the magistrate to pay fifteen guineas costs; no fines were imposed.

The Registrar of the Architects' Registration Council has written to "The Builder" as follows: "The case is important from our point of view because it is a judicial decision that an affix containing an abbreviation of the word 'architect' may constitute an offence, and confirms the advice we have received that it would be an offence for an unregistered person to put the affix A.R.I.B.A. after his name even were he a member of the Royal Institute."

New Tunnel under the River Thames.

THE main contract for constructing the Dartford–Purfleet tunnel under the river Thames has been let to Messrs. Edmund Nuttall, Sons & Co. (London), Ltd. The total cost is estimated at more than £10,000,000, of which the driving and lining of the tunnel and the construction of its immediate approaches will account for about £6,000,000. The tunnel will accommodate two lines of traffic. It will be 4700 ft. long and there will be 1500 ft. of cuttings at the approaches. At the lowest point in the tunnel the road will be 100 ft. below high-water level. It is expected that the work will take about five years to complete. The Ministry of Transport and Civil Aviation has appointed Messrs. Coode & Partners as consulting engineers, and Messrs. Mott, Hay & Anderson will act as executive engineers.

The main tunnel will be 28 ft. 2 in. diameter inside the lining of flanged cast-iron segments, varying between 1 ft. 6 in. and 2 ft. 6 in. in width and weighing about 1 ton, according to the ground to be supported. The flanges of the segments, which will be 14 in. deep and up to 2½ in. thick, will be bolted together with steel bolts of 1½ in. diameter. As the lining is erected it will be grouted to the ground by cement injected under pressure, and the joints between the segments will be caulked with lead.

The tunnel will be entirely in water-bearing ground, consisting of silt and gravel overlying chalk. The work will be done in compressed air. On each side of the river there will be three compressors, each with a capacity of 5000 cu. ft. of air per minute and driven by 750-h.p. motors. some additional compressors will be installed for use in emergency, and subsidiary plant will be provided for supplying air for pneumatic tools. Separate air-locks will be provided for men and materials.

The tunnelling will be carried out with the aid of shields weighing nearly 300 tons and divided into sixteen working compartments. The shields will be moved forward by forty-two hydraulic rams mounted inside the shield and bearing on the last ring erected. The rams will operate at a maximum pressure of 6000 lb. per square inch, and the shield will be able to exert a total thrust of over 6000 tons. Secondary hydraulic rams, operating at a lower pressure, will be used for supporting the face of the tunnel and other purposes, including hydraulic machines for lifting the segments and placing them in position.

Two of the shields for the main tunnel were erected in the shield chambers built on each side of the river when the pilot tunnel was driven in the year 1938. These shields will be driven a distance of 2800 ft. under the river. When the first shield has moved away from the chamber on the Kent side, another shield will be erected in the same chamber and driven in the landward direction towards the tunnel entrance. On the Essex side the landward drive is relatively short, and it is possible that a shield will not be used.

The pilot tunnel is 12 ft. internal diameter, and is situated within the line of the main tunnel, which will, in effect, be an enlargement of the excavation made for the pilot tunnel. The cast-iron segments of the pilot tunnel will be recovered as the work proceeds. The stabilisation of unstable ground at the site of the tunnel will be carried out from the pilot tunnel.

The carriageway in the tunnel will be 21 ft. wide between the kerbs and will be carried on a reinforced concrete structure designed to carry the standard Ministry of Transport loading. The minimum vertical headroom above the roadway will be 16 ft. at the kerb. Pedestrians will not be admitted to the tunnel.

Road Improvement at Ilford.

WORK has started on the reconstruction and widening of part of Eastern Avenue at Ilford, Essex, at a cost of £274,000. The works include the demolition of part of Newbury Park railway station and the replacement of the existing bridge over the railway with a three-span reinforced concrete structure 72 ft. long. There will be two carriageways each 30 ft. wide and footpaths, and pedestrian subways to the railway station and a hospital. The works have been designed for the Ministry of Transport and Civil Aviation by Messrs. Rendel, Palmer and Tritton, consulting engineers. The contractors are Messrs. W. & C. French, Ltd.

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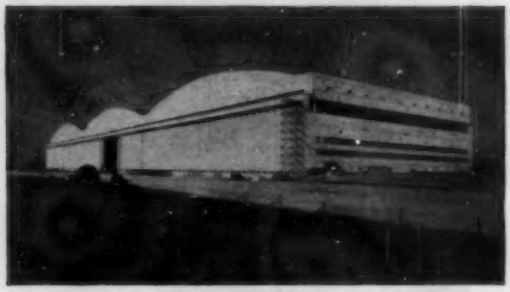

Apron and Link Road.

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Book Reviews.

"Uber die Biegungsmomente der Randtrager von Zreuzweise bewehrten Fahrbahnplatten", by Adolf Pucher. **"Das durch elastische Spanten versteifte, beliebig gelagerte Rohr"**, by Ernst Gruber. Bautechnik-Archiv, Heft 10. (Berlin: Wilhelm Ernst & Sohn. Price 9.80 D.M.)

BAUTECHNIK-ARCHIV gives research papers that are considered to be beyond the scope of a technical journal.

Dr. Pucher deals with the calculation of bending moments on edge-beams supporting slabs spanning in two directions. The design of such beams can be simplified by the use of influence surfaces, represented in the form of contour lines. The author has calculated the influence field for the bending moment at midspan of a rigid edge beam ($I \to \infty$) supporting a rectangular slab having a length to

TABLE I.

LOADING	EXACT METHOD BY PUCHER		APPROXIMATE METHOD BY LOESER ACCORDING TO GERMAN CODE NO. 1045
	FIXED BEAM ($I = \infty$)	SIMPLE BEAM FOR COMPARISON, WITH $B = \frac{4}{7}L$	
UNIFORMLY-DISTRIBUTED LOAD, p	$M = 0{\cdot}0238\,pL^2$	$M = 0{\cdot}0454\,pL^2$	$M = 0{\cdot}0553\,pL^2$
CONCENTRATED LOAD P AT MID-SPAN	$M = 0{\cdot}090\,PL$	$M = 0{\cdot}0857\,PL$	$M = 0{\cdot}100\,PL$

breadth ratio of 10/7. For slabs that are nearly square with slender longitudinal beams it is necessary to consider the stiffness ratio of the beams and slabs to secure beams of the minimum size. The elastic yield of the beam is treated as a disturbance factor to the primary case of rigid beams.

The results are given in *Table* I, where they are compared with the approximate method of Loeser which is permissible under German Code of Practice No. 1045. This table gives the bending moment at midspan of an edge-beam supporting a slab 10 m. long by 7 m. wide. The author concludes that the method based upon the lever-arm principle according to the German code does not always give correct results. This view seems, however, to have little significance for transverse beams in bridge decks, which in any case must be designed for lateral stiffness resulting in larger beams than would be needed to carry the live load. The paper deals with simple cases, but points out that the principle can be applied to more complicated cases.

Dr. Gruber deals with shells with a conical median surface stiffened with elastic hoops, such as occur in high masts, pipes, cooling towers, and containers for gas and liquids. The author considers first a truncated cone-shaped membrane, without stiffeners, firmly supported at one end where it cannot deform freely and cantilevered at the free end. To this is applied the influence of the elastic stiffeners, and equations are derived for stresses in the combined systems. The derivations require a thorough knowledge of higher mathematics, including the Fourier series.

"Versuche zur Festigkeit der Biegedruckzone." By H. Rusch. Deutscher Ausschuss für Stahlbeton No. 120. (Berlin: Wilhelm Ernst & Son. 1955. Price 14 D.M.)

THIS is a study of the strength of reinforced concrete beams related to the strength of unreinforced prisms subjected to combined bending and compression. The ordinary theory of bending of beams is based upon a simplified assumption of elasticity, but deformations may be considerably increased by creep and plasticity. The method is based upon the conception of the compressive zone of a beam in bending being represented by the eccentric load acting on the prism (see sketch).

The author describes tests of prisms made of ordinary grades of concrete loaded for short periods. Five prisms of each grade were subjected to varying eccentricities, and the deformations observed. In evaluating the results, the eccentricity that produced zero strain at the edge farthest from the load was determined graphically. Under such loading the prism is assumed to give a picture of the compressive zone in bending, the characteristics being clearly defined by the load, the centre of gravity of the stress diagram, and the compressive strain at the edge. The results are plotted in diagrams and indicate characteristics of the concrete within the limits of ideal plasticity and elasticity.

With weaker grades of concrete there is a tendency to approach the plastic state, that is a rectangular distribution

of stress, while for stronger concretes elastic conditions are approximated, that is a triangular distribution of stress. The tests give some indication of the distribution of stresses in the compressive zone of a beam but the author suggests that further tests with long-period loading are necessary.

"Beton im Seewasser." By A. Hummel and K. Wesche. (Berlin: Wilhelm Ernst & Son. Price 5 D.M.)

THIS is supplementary to Bulletin No. 102 of the German Select Committee for Reinforced Concrete, and describes additional tests on the properties and strength of concrete in salt water. Prisms sawn from the original specimens were tested after two years' immersion in salt water. Generally the strength increased or remained unaltered.

"Sistemi Elastici Piani." By A. Linari. Two volumes. (Naples: Pellerano-Del Gaudio. 1956. Price 3000 lire.)

THE method of calculating bending moments on statically-indeterminate structures described is based on slope-deflection equations, but by algebraic manipulation and the tabulation of certain functions the labour involved in the solution of the equations is reduced. Nevertheless, for continuous beams and other members in which the joints are subjected to rotation only there appear to be little advantage, other than for demonstrating the universality of the method, in calculating bending moments by an indirect method of computing first the rotation of a joint and then the bending moment, when there are available methods that enable bending moments to be calculated directly. For Vierendeel girders and frames in which the joints may move horizontally or vertically as well as rotate, the "direct" methods, such as moment distribution, are not so advantageous because simultaneous equations have still to be solved; but even in these cases to deal in terms of the bending moment rather than in terms of little practical significance appears to the reviewer to be an approach most likely to be useful. In the first volume the method is derived and applied to a variety of beams and frames; the second volume consists entirely of tables of properties of members. Together they will no doubt enable those who master their contents to become competent in the analysis of statically-indeterminate structures, but it is unlikely that the methods will be popular in this country. —J. E. G.

"Beurteilung der Tragfähigkeit schwerbelasteter Strassen durch den Plattendruckversuch." By P. Siedek and R. Voss. (Berlin: Wilhelm Ernst & Son. 1956. Price 16 D.M.)

THIS report, issued on behalf of the German Institute for Road Research, describes tests made to assess the strength of roads carrying heavy traffic and constructed with a bituminous top course and a base course on a foundation of pitching. The damage could often be seen by inspection, but the plate-bearing test was used to determine the carrying capacity of each layer composing the road. The apparatus consisted of a circular plate in contact with the layer to be tested, and which was pressed downwards by a jack bearing against the chassis of a laden 12-tons lorry. The load was applied in stages at up to twice the wheel-load, and the penetration measured at each stage. Samples from undisturbed surfaces were tested in accordance with the California bearing-ratio method. The test loads caused damage in every case where the drainage and subgrade were not entirely satisfactory. The results indicated that where the road was loaded to the limit of its strength even the smallest defect in any part led to severe damage.

Books Received.

"Some Useful Hints in Practical Surveying." By K. M. Bell. 64 pages. [Richmond (Surrey): Association of Engineering and Shipbuilding Draughtsmen. Price 3s.]

"Practical Problems in Soil Mechanics." By H. R. Reynolds and P. Protopapadakis. 2nd Edition. (London: Crosby Lockwood & Co., Ltd. Price 18s.)

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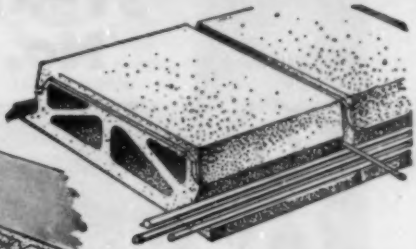

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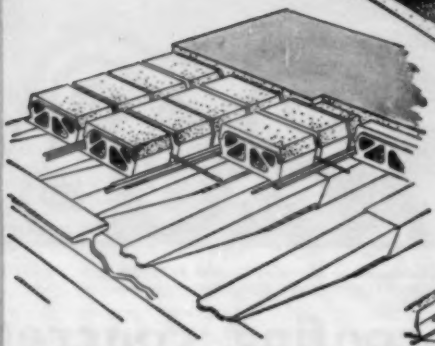

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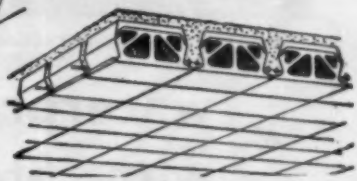

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Some views of the open-air swimming pool at he Skegness Holiday Camp. By kind permission of Messrs. Butlins Ltd.

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Resistance to Shearing of Reinforced Concrete Beams.

An investigation * has recently been carried out at the University of Illinois to review and correlate the results of previous research on shearing and diagonal tension in reinforced concrete beams, to determine the modes and characteristics of failure due to shearing, and to establish a general expression for the strength of reinforced concrete beams subjected to shearing stresses under different conditions of load. The investigation was limited to members subjected to combinations of shearing and bending forces only. A basically new empirical equation was derived for the resistance to shearing of simply-supported rectangular beams without web reinforcement and under one or two symmetrical concentrated loads. It is shown that the basic equation can be interpreted with the aid of the ordinary theory of failure in compression.

Simply-supported Beams.

Considering failures due to shearing as failures due to compression, an equation has been derived empirically to represent with good accuracy the shearing strength of rectangular simply-supported beams without web reinforcement and under one or two symmetrical concentrated loads. This equation is based on the results of fifteen different investigations of 106 beams which failed by shearing. These beams were tested over a period of forty-three years and had wide variations in their physical properties. It was found that the use of reinforcement to resist shearing increased the shearing strength of a beam more than would be accounted for by the internal forces in the stirrups.

The basic empirical equation was extended to include beams with web reinforcement, and the amount of web reinforcement required to prevent failure due to shearing was determined. Furthermore, the same equation was modified to apply to single-span T-beams and restrained beams under symmetrical concentrated loads. It was found also that the basic equation could be used to determine the resistance to shearing of a reinforced concrete beam under uniform load and, possibly, under any type of distributed load.

Excluding failure of bond, a reinforced concrete beam can fail either in bending or by shearing. Failures due to bending can be initiated either by the yielding of the tensile reinforcement or by the crushing of the concrete in the top of the beam, depending on the physical properties of the beam. Since the strength of a beam subjected to bending can be determined accurately, the purpose of this analysis was to find the amount of shearing reinforcement necessary to cause a beam to fail as a result of bending rather than of shearing.

Equations were derived which showed that a given amount of reinforcement to resist shearing increased the shearing strength of a beam in proportion to its strength without reinforcement to resist shearing rather than by an amount determined solely by the physical properties of the reinforcement. It appears that, by resisting the extension and widening of diagonal cracks, the presence of reinforcement to resist shearing increases the available compressive area of the concrete and, conceivably, restricts the concentration of the compressive strain of concrete in the region of the main diagonal crack.

Simply-supported beams under one or two symmetrical concentrated loads develop only one main diagonal crack under an applied load and fail at that section. In restrained beams, shearing and bending conditions are such as to permit the formation of more than one main diagonal crack. The beam may fail at any of these cracks, depending on the magnitudes of bending moment and shearing force and the arrangement of the reinforcement. It was found that whenever the possibility of failure of bond was precluded, the shearing strength of a restrained beam was determined by the same equation as that of a simply-supported beam. The critical section was the section of maximum bending moment.

When the longitudinal reinforcement was cut off at a section, a sudden and complete failure of the bond was possible

* "Strength in Shear of Reinforced Concrete Beams." By Armas Laupa, Chester P. Siess, and Nathan M. Newmark. (University of Illinois Bulletin No. 424. Price $1.)

by pulling out the cut-off reinforcement. This type of failure was outside the scope of the investigation and was not examined in more detail. Evidently, this is a question of bond characteristics of the reinforcement bars and the length of embedment from a diagonal crack to the end of the bar.

Restrained beams with continuous top and bottom reinforcement may have another mode of failure. Under certain conditions, as when the distance between a support and a load is short relative to the effective depth of the beam, local failure of the bond may take place in the region of high bond stress between the sections of positive and negative moments. As a result of local destruction of bond, both the top and bottom longitudinal reinforcement are in tension at a certain section. This redistribution of the internal forces results in a reduced shearing strength of the beam. Assuming that the entire tensile force were transmitted from one section to the adjacent section, an equation has been derived to represent the shearing strength for this type of failure.

Since failures due to shearing result in sudden and complete destruction of a structure, it is desirable that beams should be designed so that any failure will be in bending. In order to determine the amount of shearing reinforcement necessary to ensure failure in bending, the capacity of the beam subjected to bending should be determined first and the corresponding loading considered as an applied loading. Then, both the applied moment and shearing moment should be determined for sections where shearing is critical. The ratio between the two bending moments can be used to determine the amount of shearing reinforcement required.

Strength in Bending of Prestressed and Reinforced Beams.

THE results of tests on the bending strength of prestressed and reinforced concrete beams are described in a paper presented by Mr. Jack R. Janney, Mr. Eivind Hognestad, and Mr. Douglas McHenry at the annual convention of the American Concrete Institute held in February last.

Test results of nineteen rectangular beams are given, including (1) three with pre-tensioned steel, (2) three with grouted post-tensioned steel, (3) five with unbonded post-tensioned steel, (4) three with unbonded post-tensioned steel with deformed bars added, and (5) five with different amounts of deformed mild steel bars. All the beams were 6 in. by 12 in. in cross section and were loaded at the third-points of a 9-ft. span. The compressive strength of the concrete was about 5500 lb. per square inch and the prestressing steel was a seven-wire strand $\frac{3}{8}$ in. diameter producing an effective prestress of about 120,000 lb. per square inch. The percentage of steel varied from 1·20 to 4·75 in the reinforced beams and from 0·322 to 0·965 per cent. in the prestressed beams. The conclusions are as follows.

Calculations of ultimate strength including equations for the equilibrium and the compatibility of strains permitted a satisfactory prediction of the effects of all important variables on flexural strength.

The strengths of the prestressed beams with pretensioned steel and the corresponding beams with bonded post-tensioned steel were nearly equal and 20 per cent. to 40 per cent. higher than the strength of the corresponding beams with unbonded post-tensioned steel. Unbonded beams tested in which one strand was replaced by embedded deformed bars had ultimate strengths near to those of corresponding prestressed beams with bonded steel.

Deflections at mid-span were smaller in the case of prestressed beams than in the case of reinforced beams at the same working load, but at loads near the ultimate the reverse was true. Over the entire range of loading the prestressed beams were more elastic than the reinforced beams with respect to recovery after removal of the load.

The force in a pre-tensioned rusted strand of $\frac{3}{8}$ in. diameter was transferred to concrete with a strength of 4000 lb. per square inch in a length of not more than 12 in. The performance of the beams with pre-tensioned steel and with post tensioned grouted steel indicated that the bond between the grout and the steel was as satisfactory as that between embedded steel and concrete.

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Salary £1,000 to £1,250 according to experience.

Applicants should send details of qualifications and experience to

UDALLS, West Quay Road, Southampton.

REINFORCED CONCRETE DETAILERS REQUIRED

Consulting Structural Engineers have two vacancies in their Surrey office for Reinforced Concrete Designer-Detailers. Applicants must be experienced in detailing all types of reinforced concrete structures, and especially multi-storey building frames.

Commencing salaries will range from £700 to £900 per annum, depending on experience and ability. The positions offered are permanent, with excellent prospects for advancement. A Staff Superannuation Scheme is in operation.

Apply, giving age and full details of experience, to BOX No. 4273, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

MIDDLESEX COUNTY COUNCIL
COUNTY ARCHITECT'S DEPT.

STRUCTURAL ENGINEERS, APT V (£795–£970 per annum, plus weighting). Should be experienced in design and detailing of framed structures in reinforced concrete and structural steelwork and supervision of draughtsmen. JUNIOR STRUCTURAL ENGINEERING ASSISTANTS (£595–£675), plus weighting. To assist in detailing of framed structures in reinforced concrete and structural steelwork and preparation of steel schedules. Commencing salaries according to ability and experience. London weighting additional £30 at 26 years and over, £20 at 21–25 years. 5-days' week of 38 hours. Established and pensionable. Prescribed conditions. Application forms (stamped addressed foolscap envelope) from COUNTY ARCHITECT, 1 Queen Anne's Gate Buildings, Dartmouth Street, London, S.W.1, returnable by 22 June. (Quote S.840.) Canvassing disqualifies.

SITUATION VACANT. A.M.I.Mech.E., with some electrical and building constructional experience, or A.M.I.C.E., with mechanical or electrical experience, required for interesting position in the Middle East. Age preferably 30–35. Basic salary according to experience, plus free furnished quarters and living allowance. Apply, with full details, to BOX 4281, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATIONS VACANT. Experienced reinforced concrete detailers required. Prospect of contemporary design experience. Good salaries. Five-days' week. Holiday arrangements honoured. JOHN DE BREMAEKER & PARTNERS, 3 Southampton Place, London, W.C.1. Chancery 3785.

SITUATION VACANT. Design assistant required immediately for company manufacturing prestressed concrete. Knowledge of prestressed concrete design not essential. Write, giving details of qualifications, experience, and salary required, to CHARLTON CONCRETE CO., LTD., 7A Pulteney Street, Bath, Somerset.

SITUATIONS VACANT. Reinforced concrete or steelwork draughtsmen and designers, age 21 to 35, wanted in medium-size consulting engineers' office, Westminster, where specialisation is avoided and opportunity exists to learn all aspects of civil and structural engineering. Five-days' week. Holiday arrangements respected. BOX 4282, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Experienced reinforced concrete detailer-draughtsman required by ROM RIVER CO., LTD. Permanent and progressive post. 36½-hours' week. Superannuation scheme. Salary according to age and experience. Existing holiday arrangements honoured. Telephone Euston 7814.

SITUATIONS VACANT. Reinforced concrete designer-draughtsmen and detailers required for work on a variety of structures, including shell roofs. Apply, giving full particulars, to JAMES E. WARDROPPER, CONSULTING ENGINEER, 116 Victoria Street, London, S.W.1.

PORT OF LONDON AUTHORITY

The Port of London Authority have vacancies in their Drawing Office at Head Office, Trinity Square, London, E.C.3, for Leading Technical Assistants for reinforced concrete and steelwork design and detailing under supervision. The salary scale is £600 by £25 to £725 per annum. There is an increment bar at £650 per annum, progress beyond which will depend on service, ability, and qualifications. The commencing salary will depend on qualifications and experience. Candidates must possess the Ordinary National Certificate or equivalent in appropriate subjects.

Applications giving age, qualifications and full details of experience should be addressed to the ESTABLISHMENT OFFICER, PORT OF LONDON AUTHORITY, Trinity Square, London, E.C.3.

WEST SUSSEX COUNTY COUNCIL
County Architect's Department

Applications are invited for the following appointment :

ASSISTANT STRUCTURAL ENGINEER at a salary in accordance with A.P.T. Grade VI (£880 to £1,080). Commencing salary according to experience.

Further particulars should be obtained from the County Architect, County Hall, Chichester, to whom detailed applications must be submitted not later than the 25th July, 1956.

T. C. HAYWARD
Clerk of the County Council

County Hall,
Chichester.
28th May, 1956.

(*Continued on next page.*)

MISCELLANEOUS ADVERTISEMENTS.

(Continued from previous page.)

SITUATIONS VACANT. Design engineers, draughtsmen, and detailers, with experience in reinforced concrete or structural steelwork, required urgently for permanent position in consulting engineers' offices in London and Leeds. Steady expansion of firm provides opportunities for suitable candidates to undertake responsible posts covering all aspects of design work, or to broaden their experience and capabilities, on a generous salary scale. Box 4280, CONCRETE AND CONSTRUCTIONAL ENGINEERING, Dartmouth Street, London, S.W.1.

SITUATIONS VACANT. Design-detailers and detailers experienced in reinforced concrete required for London professional office. Knowledge of steelwork an advantage but not essential. The work is varied and offers scope for initiative and advancement. Opportunity for site visits, and possibility of joining resident engineer's staff later on. Five-days' week. Apply, stating experience and salary required, to FARMER & DARK, Romney House, Tufton Street, London, S.W.1.

SITUATION VACANT. Senior designing engineer urgently required by consulting engineers in London. Capable of taking charge of reinforced concrete projects. Permanent position, interesting work. Box 4279, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATIONS VACANT. Senior and junior designers for reinforced concrete and/or steelwork required for London office. Considerable scope for advancement. Juniors assisted in study courses involving day attendance. Consideration given to holiday arrangements. Box 4286, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

CHIEF OF TESTING LABORATORY

Executive required as Chief of Testing Laboratories, fully experienced in concrete and other building materials. Salary according to qualifications. Housing on outskirts of London available. Exceptional prospects for right man. Fullest details of experience and qualifications will be treated in confidence. Box, 4285, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION WANTED.

SITUATION WANTED. Structural engineer designer-draughtsman, experienced in reinforced and prestressed concrete especially and all building methods, seeks change to South coastal area from London. Box 4278, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

BUSINESS WANTED.

WANTED. Advertiser wishes to purchase established concrete products business in London or Greater London area. Box 4283, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

PARTNERSHIP.

PARTNERSHIP. Working directorship in return for additional capital not less than £15,000 for forming separate company to take over substantial and growing contractor's subsidiary specialising in design and manufacture of precast and prestressed floors and beams. Modern works N.W. area and Midlands. Alternatively, would consider joint company with another general contractor or reinforcement supplier. Principals only. Apply Box 4272, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

FOR SALE.

FOR SALE. Steel tubes and fittings. Sizes to 10 in. E. STEPHENS & SON, LTD., Bath Street, London, E.C.1. Clerkenwell 1731.

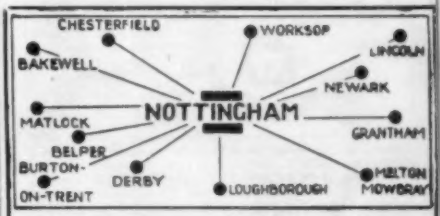
FOR SALE. Steel plates and sections to size. Tubes, fittings, etc. E. STEPHENS & SON, LTD., Bath Street, London, E.C.1. Clerkenwell 1731.

FOR SALE. Approximately 50 tons of high tensile steel rods of various diameters, mostly about ½ in. Also large tonnage of EN 8 steel, flat and square bars, in small and medium sections. WOODFIELD & TURNER LTD., Burnley. Telephone: 78134.

FOR SALE. Reinforcing bars in stock, Surrey, at £45 a ton. Immediate disposal of 40 tons of ⅞ in. and 130 tons of ½ in. FROMSON CONSTRUCTION CO., Byfleet, Surrey. Telephone: Byfleet 2361.

FOR HIRE.

FOR HIRE. Lattice steel erection masts (light and heavy), 30 ft. to 150 ft. high, for immediate hire. BELLMAN'S, 21 Hobart House, Grosvenor Place, London, S.W.1.



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